# ACI News Letter

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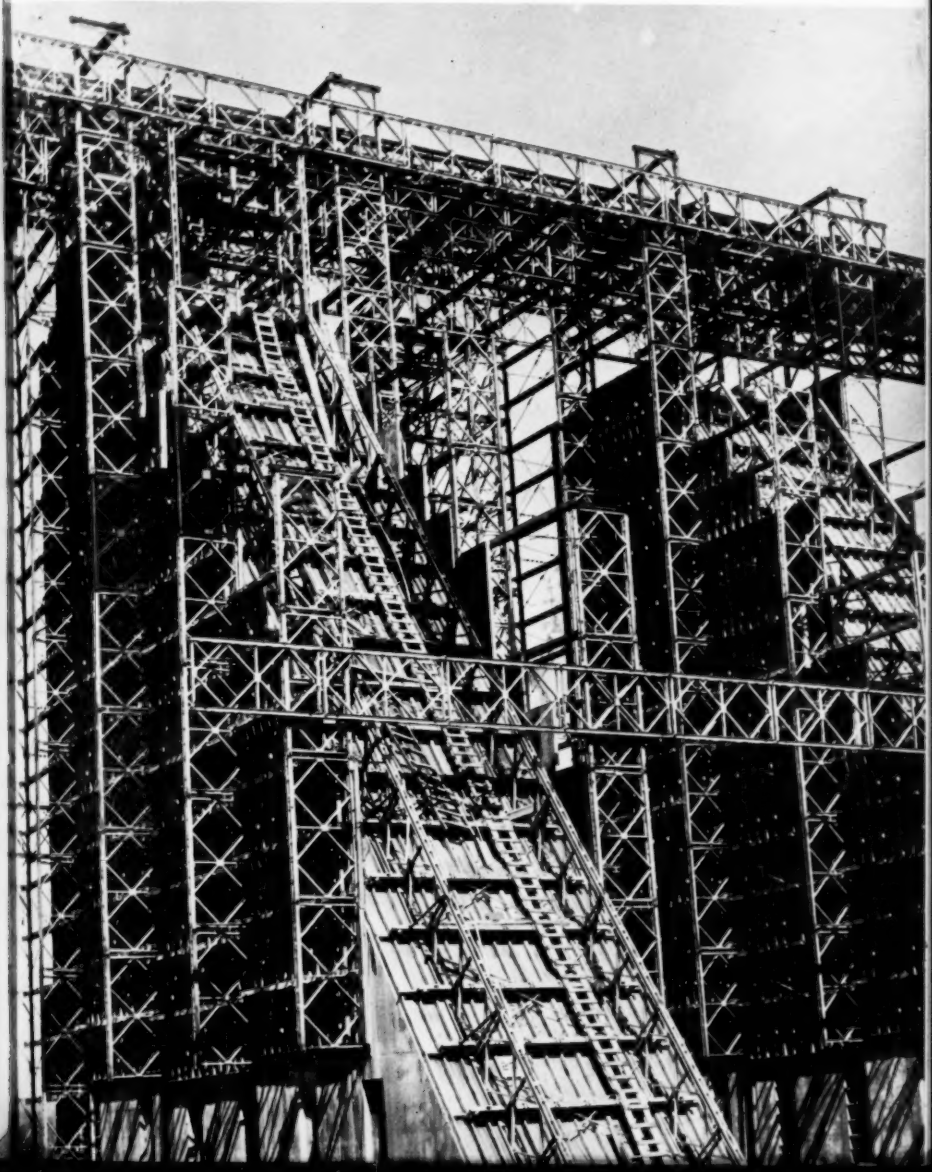

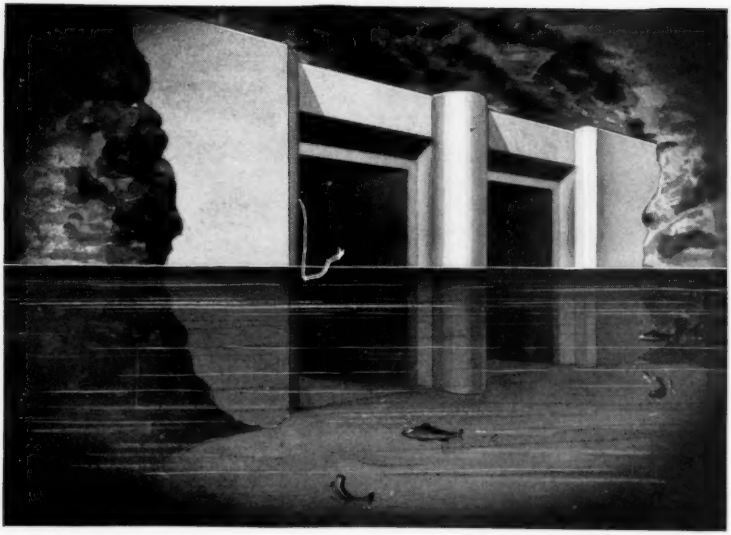

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# ACI

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# Jackson named ACI president; Richart honored at 46th Annual Convention

With an attendance of 609, the 46th Annual Convention in Chicago, February 20-22, gained eighth place in the attendance records for ACI conventions. The record attendance is still held by the 44th Annual Convention in Denver in 1948 with an all-time high of 886.

Frank H. Jackson, Principal Engineer of Tests, Bureau of Public Roads, Washington, D. C., became president of the Institute for the current convention year. He succeeded Herbert J. Gilkey, Head of the Department of Theoretical and Applied Mechanics, Iowa State College, Ames, Ia.

Other officers elected at the convention included A. T. Goldbeck, Engineering Director, National Crushed Stone Assn., Washington, D. C., vice-president and A. E. Cummings, I. L. Tyler, Charles S. Whitney and C. E. Wuerpel, directors.

The annual noon luncheon, with 326 attending, featured the presentation of tokens of ACI awards to seven men and the introduction of the new officers by President Gilkey. Frank E. Richart, C. P. Siess, N. M. Newmark, T. M. Kelly, L. Schuman, F. B. Hornibrook and S. O. Asplund were honorees. The address of the retiring president was "The Zig-Zag Course of Concrete Progress" (p. 573 this JOURNAL).

In honoring Professor Richart with the first Alfred E. Lindau Award—"for his years of labor toward the improvement of reinforced concrete design practice" —President Gilkey said that it was most fitting that the first Lindau Award go to a man who had been so closely associated with Alfred Lindau.

The award was founded in 1947 by the Concrete Reinforcing Steel Institute in honor of the late Alfred E. Lindau. It is given only for outstanding contributions to reinforced concrete design practices and is not mandatory each year.

The three-day meeting at the Edgewater Beach Hotel featured sessions on inspection; ACI Building Code studies; structural design of concrete pavements; prestressed, precast and reinforced concrete; admixtures; research and a panel discussion of current concrete problems.

By holding concurrent sessions, the evenings were left free for committee meetings and get-togethers, and it was possible to present a greater variety of papers of interest to the members. More than 100 participants, and the presentation of 30 papers and

many discussions, made it evident that concrete knowledge is still expanding and that the need for more information is far from satisfied.

Sixteen committees took advantage of the free time to hold meetings, with outstanding attendance by both committee members and visitors.

The day following the close of the general sessions, the local committee under the chairmanship of Harry Delzell, arranged for a tour through the new Portland Cement Assn. laboratory in Skokie and the Technological Institute of Northwestern University in Evanston.

The local committee also included Frank W. Edwards, Harrison F. Gonnerman, William T. F. Hooper, Jr., John F. Seifried, C. P. Siess, H. F. Thomson and W. W. Wallace.

# 46th Annual Convention

## Inspectors Are Not "Private-Eyes"

The inspector is a vital part of any construction project and he is on the job not only to protect the interests of the owner, but also to be of assistance to the contractor.

These points were emphasized at the opening session of the 46th Annual Convention Monday afternoon. J. W. Kelly, University of California, Berkeley, Calif., was session leader.

### The inspector

"The inspector must see that the plans and specifications are fulfilled and translated into good construction. He must also be a coordinator, expediter and adviser. He is not a detective to 'catch' the contractor," said Miles N. Clair, vice-president of Thompson and Lichtner Co., Inc., Boston, Mass. Mr. Clair also said that great improvements in concrete will come with recognition of the importance of competent, well trained inspectors.

### Inspection and testing of materials

Nicolaas T. F. Stadtfeld, Division Engineer, Board of Water Supply, City of New York, stated that the inspection of cement, aggregates and other materials at the manufacturer's plant before shipping to the job-site helps in getting better materials for better concrete. He described the inspection procedures of the New York Board of Water Supply and stressed the necessity for manufacturing control of the cement clinker.

### Inspection of building construction

"Other factors besides just inspection must be considered to secure the best possible results in the construction of any building," said Leonard E. Dunlap, Carr and Wright, Inc., architects and engineers, Chicago. He said that these included complete and accurate drawings and specifications and the assignment of a competent and experienced superintendent to represent the architect and engineer in the field. Mr. Dunlap emphasized that in most cases the procedures in construction operations are the contractor's responsibility, not the inspector's.

### Illinois highway inspection

"Inspection and Control of Concrete for Highway and Bridge Construction" by H. W. Russell, Engineer of Materials, Illinois Division of Highways, Springfield, Ill., summarized the inspection procedures and personnel assignments used by the Illinois Division of Highways. The Division assigns full time porportioning engineers to control the concrete mixtures for pavements and large structures. He also described the mobile testing unit used by the proportioning engineers.

**A 10-minute break is taken on the inspection trip by (l. to r.) H. F. Gonnerman, Assistant to the Vice-President for Research and Development, Portland Cement Assn.; President Frank H. Jackson, Bureau of Public Roads; Retiring President Herbert J. Gilkey, Iowa State College; Institute Secretary, Harvey Whipple; and A. Allan Bates, Vice-President for Research and Development, Portland Cement Assn.**

### Inspection of mass concrete

Lewis H. Tuthill, U. S. Bureau of Reclamation, Denver, Colo., said in his paper: "Inspection of any kind can be no more effective than that permitted by the specifications, particularly, by the established job standard of inspection." Factors which influence the effectiveness of inspection include specifications, management policy, ability of the contractor and supervision of the inspector.

"Where effective inspection is wanted, management must see that the specifications and the setup of the job are such that inspection can be effective," Mr. Tuthill concluded. In the author's absence, the paper was presented by Robert F. Blanks, U. S. Bureau of Reclamation, Denver, Colo.

### The contractor's viewpoint on inspection

"There are many situations where the inspector can assist the contractor without in any way jeopardizing the owner's interests. Such instances.....go a long way toward promoting good personal relationships between the two parties and the work benefits," was a statement made in a paper by Donald C. Andrews, Turner Construction Co., New York, and Nomer Gray, Walter Kidde Constructors, Inc., New York. The authors went on to say that cooperation is a two-way street and that it is important that the contractor, inspector and owner work as a team. The owner, they concluded, should select his inspector with the same care and consideration that the contractor selects a superintendent.

In the absence of the authors, J. W. Kelly, University of California, Berkeley, Calif., presented the paper.

### Inspection of ready-mixed concrete

E. L. Howard, Pacific Coast Aggregates, Inc., San Francisco, Calif., stated that the great variety of concrete mixes produced, sometimes as many as 48 different mixes in one week, add to the problems of inspection and control of ready-mixed concrete; it is therefore important that suitable testing and control be exercised at the source of the materials. The problems also vary with each plant because of the varied equipment used.

### Inspection summary

The inspection session was summarized by Roderick B. Young and Wilfrid

Schnarr, Assistant Director of Research and Concrete Control Engineer, respectively, Hydro-Electric Power Commission of Ontario, Toronto, Ont., Canada. They stated that although the various authors differed in the emphasis they placed on the phases of inspection, they all agreed on such points as the qualifications of an inspector, his basic duties and the responsibility of his employer. They said that the inspector's basic responsibilities are substantially the same on any job, but that his actual duties vary greatly with the type and size of the construction project.

### Houses of magic

In the discussion period following the presentation of papers, Robert F. Blanks, U. S. Bureau of Reclamation, Denver, Colo., said that the production of uniform concrete depended largely on the production of uniform ingredients. Production of uniform aggregate has taken long strides. In speaking of proportioning, he added that modern batching and mixing plants have been called houses of magic because of the automatic way in which they accomplish the job.

Mr. Blanks said there hadn't been the comparable progress in the development of mixing equipment that is found in processing and batching equipment; and concluded with a needling statement that as far as control work in concrete is concerned, the production of uniform cement is probably the least controlled of any of those operations.

### A selling job

R. C. Sandberg, Rock Island, Ill., advocated changing the name "inspector" to "architect's or engineer's representative" to improve the attitudes of inspectors and contractors, and the relationships on the job.

R. A. Lonier, Illinois Division of Highways, Chicago, mentioned that young engineering graduates don't go into the inspection field; "I think we have a selling or educational job on our hands; I'd like to see young men come up who want the job of inspection, thereby improving the quality of inspection."

Rather than use "cub engineers," R. B. Young, Hydro-Electric Power Commission of Ontario, Toronto, Ont., Canada, advocated trained technicians; that practical men, specially trained for the work and knowing their job thoroughly, would be more likely to take inspection duties as a life work than a young engineer who does inspection work as a means of gaining experience.

### A matter of experience

Carl E. Swanson, Swanson and Maiwald, architects and engineers, Moline, Ill., said that good inspection is a matter of experience, of integrity, and "a little bit of this matter of getting along with each other."

Terrell R. Harper, Dallas, Texas, discussed methods of inspection. Joseph Bayer, Boswer-Morner Laboratories, Dayton, Ohio, thought that commercial laboratories should be paid for inspection services on a basis of competence, not on a basis of competitive bidding; that competitive bidding does not encourage good pay for inspectors, therefore losing good inspectors and getting poor job results. S. J. Warberg, Giffels and Vallet, Inc., Detroit, Mich., emphasized the inspector's need for properly written specifications.

### Ready-mix control

The control of ready-mixed concrete was discussed by several members, including Harry F. Irwin, Warner Co., Philadelphia, Pa., who said that their firm uses radio-equipped automobiles to get word from the job-site to the plant about job conditions so that adequate control could be maintained.

"We need the cooperation of the owner; we need the cooperation of the engineer in writing specifications which we can fulfill; and then the industry can produce well controlled concrete," asserted Stanton Walker, National Ready Mixed Concrete Assn., Washington, D. C.

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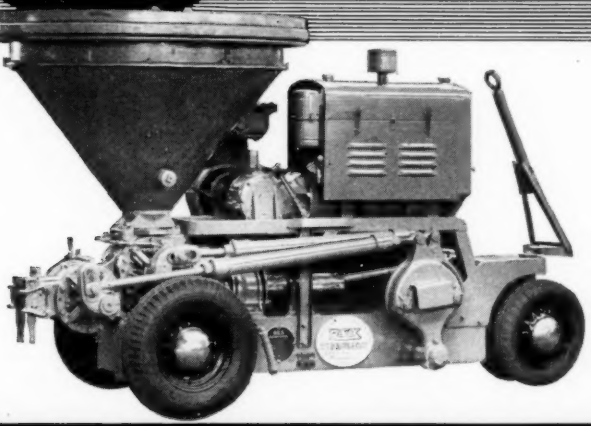

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# ACI Building Code Studies

Concurrent sessions were held at Edgewater Beach Hotel Tuesday morning, February 21. President Herbert J. Gilkey served as chairman of the session on ACI Building Code studies and committee problems in the Ballroom. The first part of the session was under the leadership of Chester L. Post, chairman of Committee 318, Standard Building Code.

### Deformed reinforcing bars

"The new-style deformed reinforcing bars with higher, closely-spaced deformations have, in a number of test programs, proved their superiority in bonding to concrete, in forming a series of infinitesimal, closely-spaced cracks instead of a few wide ones, and in providing the equivalent of 'special anchorage' as now set up," stated Raymond C. Reese, consulting engineer, Toledo, Ohio, who traced the development of deformed reinforcing bars from the original plain round or square bars to the present improved deformed patterns. He stated that these new bars improved bond resistance, permitting safe working stresses of about 10 percent of the ultimate concrete stress. The equivalent of "special anchorage" would permit higher values in diagonal tension; and the new bars give accurate crack control since the higher projections and closer spacing of deformations prevent the formation of a few large cracks and make the concrete and steel work together as a unit.

Mr. Reese said that there would be "a possible increase in tensile stresses due to more positive bonding and a continuous bonding all along the bar instead of a special anchorage at the end." He concluded that much work has been done to improve reinforcing bars, but still further progress is envisioned.

### Two-way floor slabs

Nathan M. Newmark and Chester P. Siess, University of Illinois, Urbana, Ill., proposed a new design specification for two-way floor slabs based on analyses of continuous rectangular slabs carrying a uniformly distributed load. Account is taken of the continuity of the slab, of the torsional stiffness of the beams, and of the deflection of the beams. The proposed specifications differ from those in the present ACI Code in that moment coefficients are given for average moments on sections extending the full width of a panel and the magnitudes of the coefficients are less than those currently used.

The authors said that when both the slab and the beams are considered, the proposed specification permits considerable economy over the present code provisions.

The paper was discussed briefly by L. J. Mensch, Evanston, Ill., and R. C. Sandberg, Rock Island, Ill.

## Problems in precast construction

F. N. Menefee, Professor of Engineering Mechanics, University of Michigan, Ann Arbor, Mich., was session leader for the second portion of the Tuesday morning meeting in the Ballroom. The program covered some of the problems common to Committees 208, Bond Stress; 318, Standard Building Code; 323, Prestressed Reinforced Concrete; 324, Precast Reinforced Concrete Structures and 711, Precast Floor Systems for Houses.

### Spacing of moment bars in precast joists

Tests of precast joists whose bar spacing was less than that provided for in the ACI Code were described by Professor Menefee and H. L. Kinnier, University of Virginia, Charlottesville, Va. The results indicate that $\frac{3}{8}$-in. spacing of bars, with $\frac{3}{4}$ in. maximum aggregate, will produce joists that will not fail in bond. The authors tested 14 joists.

C. D. Williams and F. Bromilow, University of Florida, Gainesville, Fla., presented supplemental information also

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# Bethlehem Reinforcing Bars

**S. O. Asplund, Stockholm, Sweden, awardee of the Institute's Construction Practice Award, was not the only cameraman in the crowd on the inspection trip through the new PCA laboratory and the Technological Institute of Northwestern University.**

on tests of 14 precast joists. All of the joists tested failed in shear, and the authors discussed the reasons for these failures. Pull-out tests were also made to study the bond properties with the elimination of other variables.

Other tests were described briefly by John Kerr Selden, University of Toledo, Toledo, Ohio, and Douglas McHenry, U. S. Bureau of Reclamation, Denver, Colo.

### Proposed specification for minimum bar spacing and protective cover

"The use of relatively small size aggregates and favorable conditions for quality control make possible closer spacing of reinforcing bars in precast concrete work than those specified or required in conventionally poured-in-place concrete construction," said A. Amirikian, Head Designing Engineer, Bureau of Yards and Docks, Navy Dept., Washington, D. C. He then recommended a new basis for specifying bar spacing and cover in proposed specifications applicable to precast concrete work.

Stephen J. Chamberlin, Iowa State College, Ames, Ia.; William T. F. Hooper, Jr., Northwestern University, Evanston, Ill.; and L. J. Mensch offered short discussions of the problem.

### Cracking in precast framing members

Mr. Amirikian presented a second paper, proposing an acceptability clause which would give practical definitions of cracks and specify limitations for acceptability. He said that the fabricator often is penalized by unreasonable rejections of cracked precast members because of the absence of definite guides for inspection and suggested that the proposed clause should eventually form a part of the specifications on precast concrete work.

### Patent status of prestressed concrete

"The principle of prestressing has been employed to improve the stability of structures from the earliest times," asserted Curzon Dobell, vice-president, Preload Enterprises, Inc., New York. He continued that the first man to conceive of curving the staves of a barrel, so as to force metal hoops around them from opposite ends, was probably the first to apply the principal to a circular structure.

"Indeed, whether she knew it or not, the first woman to wear a corset, likewise was applying external forces to neutralize undesirable bursting stresses and improve the stability, as well as the appearance, of the structure," he said. He gave a brief account of the more important groups of prestressed concrete patents and their uses.

Mr. Dobell then discussed the relationship of prestressed concrete to building codes. He mentioned several codes written in Europe, but stressed the point that since the field of prestressing is still in a fluid state, ACI should refrain from drafting a code until the techniques and methods are further advanced. He said that a code at this time might harm the further development of prestressed concrete.

Additional information was presented by A. E. Cummings, Raymond Concrete Pile Co., New York, and L. J. Mensch.

## Structural Design of Concrete Pavements

The problem of getting good concrete pavements to withstand the ever increasing wheel loads on modern highways was the subject of the other concurrent session Tuesday morning. President Jackson was chairman of the meeting in the East Lounge of Edgewater Beach Hotel. L. W. Teller, Bureau of Public Roads, as session leader, introduced the subject by summarizing the activities of Committee 325, Structural Design of Concrete Pavements for Highways and Airports.

### Influence of subgrades and bases

Kenneth B. Woods, Professor of Highway Engineering and Associate Director of the Joint Highway Research Project, Purdue University, Lafayette, Ind., said that the structural capacity of rigid pavements can be improved by location procedures which utilize the best in topographic position and subgrade soil textures. For inferior situations—in regard to position and soils—the use of base courses must be evaluated against the economy of using thicker slabs, more reinforcing, or a combination of the two. He recommended that design practices be developed by regions, where the local conditions and type and availability of materials govern, rather than a standard practice for all areas.

Mr. Woods' paper produced a good deal of discussion. Henry Aaron, Civil Aeronautics Administration, Washington, D. C., stated that the influence of subgrade soil on pavement thickness requirements is probably greater than any other single factor. He discussed the CAA practices in evaluating and correlating airport pavement performance with subgrade classification.

"Studies have proven that the impact, in a normal landing, may be entirely ignored as far as runway pavement design is concerned," declared Mr. Aaron. He said that the critical areas are taxiways, aprons and the ends of runways.

L. A. Palmer, Bureau of Yards and Docks, Navy Dept., Washington, D. C., said that the problem of "soil control," bringing about those subgrade conditions that can assure both permanency and uniformity of support, remains the most serious and difficult of all the problems involved in the design of concrete pavements.

In discussing construction practices and pavement performances in Ohio, T. J. Kauer and Charles W. Allen, Ohio Dept. of Highways, Columbus, Ohio, emphasized that one of the primary functions of subbases under concrete pavement is the prevention of pumping.

Virginia highway practices were recounted in the prepared discussion by T. E. Shelburne, Virginia State Highway Dept.; William Van Breemen, New Jersey State Highway Dept. described procedures used in that state. "The employment of granular subbases," he said, "has had two important beneficial effects. It appears to have prevented pumping and it has effected a marked reduction in cracking."

"The important point for engineers to recognize is when the subbases are necessary and when not necessary," said Leo M. Arms, Portland Cement Assn., Chicago.

### Problem of slab dimensions

In reporting the work of Subcommittee 2 of Committee 325, Structural Design of Concrete Pavements for Highways and Airports, A. T. Goldbeck, Engineering Director, National Crushed Stone Assn., Washington, D. C., said that the dimensions of concrete pavement slabs depend on and are influenced by many factors, economic as well as physical. He tabulated the factors and discussed their relationship to the problem of slab dimensions.

**Design and construction of joints in concrete pavements**

was discussed in a paper presented by William Van Breemen, New Jersey State Highway Dept., Trenton, N. J., and E. A. Finney, Michigan State Highway Dept., Lansing, Mich. They considered the fundamental conditions involved in the design and construction of pavement joints and described various types of joints and load transfer devices.

"The solution of the joint problem has not been fully attained," they said, and recommended that the general standard of workmanship be improved, that a uniformly consolidated subbase should be provided under all pavements, and that load transfer units should be evaluated on the basis of a known standard.

Thomas B. Pringle, Corps of Engineers, Dept. of the Army, Washington, D. C., supplemented the paper by a discussion of jointing practices used by the Corps of Engineers in constructing pavements for heavy wheel loads.

Other discussers included H. W. Russell, Illinois Division of Highways, Springfield, Ill., and Fred Woeller, Illinois Institute of Technology, Elmhurst, Ill.

**Use of reinforcing steel in pavements**

"Many unanswered questions remain before use and design of steel reinforcement in concrete pavements can be fully rationalized," stated Bengt F. Friberg, Chief Product Engineer, Granite City Steel Co., Granite City, Ill. He said that the survey and maintenance reports of highway departments can give information which is of value in appraising the use of steel and advancing design criteria for reinforced pavements. Mr. Friberg summarized the progress and development of the practice of using reinforcing in pavements.

The prestressed concrete pavements at the Paris International Airport were described by Phillip L. Melville, Virginia Highway Research Council, Charlottesville, Va. He stated that the prestressed pavement had worked so well that French engineers were preparing plans to construct the city streets and main boulevards in Paris following the same principle.

## Awards and Officers Announced at Annual Luncheon

More than 300 members and guests attended the noon luncheon in the Marine Dining Room of the Edgewater Beach Hotel. Retiring President Gilkey presented tokens of ACI awards to Frank E. Richart, N. M. Newmark and C. P. Siess, T. M. Kelly, L. Schuman and F. B. Hornibrook, and S. O. Asplund.

**Awards**

Professor Richart, University of Illinois, Urbana, Ill., was the choice for the first Alfred E. Lindau Award...."for his years of labor toward the improvement of reinforced concrete design practice."

Professors Newmark and Siess, University of Illinois, received the Wason Medal for "the most meritorious paper" of the year, "Rational Analysis and Design of Two-Way Concrete Slabs," which, appearing in the ACI JOURNAL, December, 1948, proposed a new method for the design of two-way building slabs.

Mr. Kelly, Corps of Engineers, Portland, Ore.; Mr. Schuman, National Bureau of Standards, Washington; and Mr. Hornibrook, Oakland, Calif., received the Wason Medal for "noteworthy research" reported in "A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions" in the ACI JOURNAL, September, 1948.

The research employed many types of aggregate, combined in varying amounts and sizes with high- and low-alkali

cements and formed into mortar bars. The expansions of the bars were measured at ages ranging from one month to four years.

Mr. Asplund, Stockholm, Sweden, who won the American Concrete Institute Construction Practice Award received as its token a bronze plaque for his paper "Strengthening Bridge Slabs with Grouted Reinforcement," ACI JOURNAL, January, 1949.

On the project described, the negative reinforcing bars settled as much as 2½ in. out of correct position. Various means for incorporating negative reinforcement at correct height in the finished structure were discussed and the method of grouting additional bars in grooves cut with the aid of a diamond saw described.

### Greetings

Mr. Asplund extended greetings of the Swedish Concrete Assn. with the assurance that ACI activities were highly regarded in Sweden.

### New officers

Frank H. Jackson, Principal Engineer of Tests, Bureau of Public Roads, Washington, D. C., was elected president of the Institute, succeeding Professor Gilkey.

A. T. Goldbeck, Engineering Director, National Crushed Stone Assn., Washington, was named vice-president for a two-year term.

New directors each for a three-year term include A. E. Cummings, Director of Research, Raymond Concrete Pile Co., New York; I. L. Tyler, Manager of Field Research, Portland Cement Assn., Chicago; Charles S. Whitney, Ammann and Whitney, consulting engineers, New York and Milwaukee; and Charles E. Wuerpel, Technical Director, Marquette Cement Mfg. Co., Chicago.

The five elected members of the nominating committee are J. F. Barbee, Harmer E. Davis, G. L. Lindsay, Harmon S. Meissner and Kenneth B. Woods. These five, serving with the three ACI presidents last past, will constitute the 1950 nominating committee. Professor Davis, receiving the most votes of the five, will serve as chairman.

*Courtesy Engineering News-Record*

**Retiring President Gilkey (left) hands the gavel to President Jackson**

### Infinity and time

Retiring President Gilkey presented President Jackson with a specially prepared test briquet as being symbolic of his past work in the cement area of the concrete field and of his future work as new president of the Institute.

"Laid on its side, it represents the symbol infinity, the period through which the argument between the briquet and the compression test has continued, and will doubtless continue," observed Professor Gilkey. "Stood on end, it represents the hour glass, a mark of progress standing for the work of the Institute. Laid flat it serves as a paper weight which again, like the Institute, may be relied upon to function continuously, effectively, and unobtrusively."

Retiring President Gilkey closed the noon luncheon with an address on "The Zig-Zag Course of Concrete Progress," which appears in full, p. 573 of this issue.

## Prestressed, Precast and Reinforced Concrete

Concurrent sessions were also held Tuesday afternoon, with Vice-President Harry F. Thomson serving as chairman of the meeting in the Ballroom.

### Corrosion protection of thin precast concrete sections

A research project being conducted at Virginia Polytechnic Institute was described by Dan H. Pletta, E. F. Massie and H. S. Robins. The authors described a new approach, using an electrical resistance technique, for measuring the rate of corrosion of steel reinforcing in thin precast members. They proposed that the protection afforded reinforcing steel by any concrete mix and any depth of cover be indicated by the time required for the reinforcing to corrode to one-half of its original cross-sectional area, and they referred to this as the half-life of the concrete. They recommended procedures for protecting reinforcement in thin precast sections based on the results of the tests.

Discussion was offered by John A. Murlin, George L. Dahl, architects-engineers, Dallas, Texas, and F. N. Menefee, University of Michigan, Ann Arbor, Mich. Professor Menefee exhibited several slides showing the force of corrosive action on steel to emphasize a plea to keep moisture from reinforcement.

### Prestressed concrete construction procedures

Prestressed concrete opens up new fields beyond those now commonly occupied said Thor Germundsson, Portland Cement Assn., Chicago. Longer and lighter bridge spans are possible by using prestressed concrete, and it permits the use of smaller and lighter beams and columns in buildings. "It can compete with plate girders and steel trusses where ordinary concrete cannot," he asserted. Mr. Germundsson reviewed the construction procedures used in the production of prestressed concrete, including methods developed by Freyssinet, Magnel, Schorer, Vobag, Hoyer and others. He also described the various long-line processes, methods of post-tensioning circular structures, and the use of thermal stretching in prestressing.

In the absence of the author, the paper was presented by D. P. Jenny, Portland Cement Assn., Chicago.

The latest work with the Schorer system was mentioned by E. J. Critzas, St. Louis, Mo. Robert A. Caughey, Iowa State College, Ames, Ia., declared that while it is not advisable to write a code for prestressing now, ACI should sponsor progress reports on the best practices for such construction; American engineers need some guidance along these lines and ACI is in a position to sponsor such reports to fill the need.

### A mathematical tool

Alfred Parme, Portland Cement Assn., Chicago, described the application of finite differences to the solution of structural problems, such as those encountered in the analysis of slabs and shells, in which the physical relationships are expressed as differential equations.

Additional comments came from Dan H. Pletta, Virginia Polytechnic Institute, Blacksburg, Va., and L. E. Grinter, Illinois Institute of Technology, Chicago.

### Precast concrete in Britain

The paper by P. G. Bowie and A. R. Collins, Cement and Concrete Assn., London, England, presented highlights of precast construction in Great Britain and Europe. They illustrated precast frames for airplane hangars, garages and farm buildings. Three systems used in housing construction, the Airey, Wates and Stent methods, were also described.

"The precast concrete industry in Britain has made great strides in the last ten years, but past progress seems of small importance compared with the

possibilities opening up before it today," they concluded.

In the absence of the authors, the paper was presented by Everett E. Ebling, Bethlehem Steel Co., Chicago.

#### Precast reinforced concrete structures

C. D. Wailes, Jr., president of the C. D. Wailes Corp., Los Angeles, Calif., focused attention on the practical application of precast construction in this country. He stated that it has been used in trestles, bridges, marine structures and many types of buildings and illustrated these with slides of some of the more outstanding structures built.

"It has proved an economical and structurally sound method," he concluded.

In the absence of the author, the paper was presented by Frank Kerekes, Iowa State College, Ames, Ia.

A. Amirikian, Bureau of Yards and Docks, Washington, D. C., cited the use of rich mixtures in precast units for marine structures to obtain concrete which is practically impervious to water or passage of moisture. The best way to obtain concrete that is tight is to use a generous amount of cement, he added.

R. E. Copeland, National Concrete Masonry Assn., Chicago, described prefabricated floor and roof units using concrete masonry units.

Additional comments were offered by Jacob J. Creskoff, Philadelphia, Pa., and J. John Brouk, Precast Slab and Tile Co., St. Louis, Mo.

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## Admixtures, Arch Roofs, Bonneville Dam

Past President Robert F. Blanks was chairman of the other concurrent session Tuesday afternoon. Recognizing the ever-increasing importance of admixtures, particularly as related to pozzolanic materials and air-entraining agents, the first part of the session was devoted to the subject of admixtures and their uses. W. T. Moran, U. S. Bureau of Reclamation, Denver, Colo., Chairman ACI Committee 212, Admixtures, was session leader for this portion of the program.

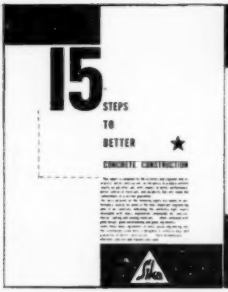
Following a brief introduction by Mr. Moran, the advantages and limitations of admixtures in the following uses were discussed: to increase resistance to freezing and thawing; to correct grading deficiencies; to counteract alkali-aggregate reaction; as integral waterproofing materials; and in concrete products.

#### Resistance to freezing and thawing

Frank H. Jackson, U. S. Bureau of Public Roads, discussed the use of admixtures to increase the resistance to freezing and thawing. He reported evidence that the use of pozzolanic admixtures, properly controlled, and in combination with air entrainment, will definitely improve durability and at the same time make possible reductions in cement content.

#### Use of admixtures in concrete products

"Admixtures have been used to a limited extent only in the manufacture of concrete products," said Bruce E. Foster, National Bureau of Standards, Washington, D. C.; "but definite advantages resulting from such use have been reported." He discussed the use of various types of admixtures in concrete products plants and the results that could be expected.

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**Correction of aggregate grading**

T. C. Powers, Portland Cement Assn., Chicago, in a paper on the use of admixtures for the correction of aggregate gradation said that most of the materials used as admixtures were mineral powders as fine or finer than portland cement.

"They therefore serve to 'correct' the properties of the cement paste," he asserted. "Since the distinction between the fine material in the paste and that in the aggregate is more or less arbitrary, mineral admixtures may also be considered as correctives for aggregate gradation."

Stanton Walker, National Sand and Gravel Assn., Washington, D. C., discussed aggregate grading and admixture relationships briefly.

**Counteracting alkali-aggregate reactions**

The most recent knowledge on this subject was briefly reviewed by Mr. Moran. He said the most reliable method now available for evaluating alkali reactivity of pozzolans is a reduction-in-expansion test.

Further comments on alkali-aggregate reactivity were by Bryant Mather, Concrete Research Division, U. S. Waterways Experiment Station, Clinton, Miss., and T. W. Thomas, University of Minnesota, Minneapolis, Minn.

**Waterproofing and dampproofing**

In reference to the use of admixtures as waterproofing and dampproofing materials, Mr. Foster said that admixtures are not a substitute for careful choice of materials and mixes, good workmanship or adequate curing, but that they should be looked upon as insurance against some of the deficiencies that occur due to deviation from good concreting practices.

**Cost of long-span concrete shell roofs**

was compared with those of timber and steel by Charles S. Whitney, Ammann and Whitney, consulting engineers, New York and Milwaukee, in the second half of the session. He said that the initial cost of a structure is not greatly affected by the choice of material, but that the annual cost of the project is different for each material. He emphasized that these are the costs to be considered at the start of a project.

Robert Zaborowski, New York, N. Y., described several other thin-shelled structures and discussed cost features of such construction.

**Effects of high-velocity water**

R. R. Clark, Office of the District Engineer, Dept. of the Army, Portland, Ore., compared the progressive erosion which has taken place in the stilling basin of Bonneville Dam with that anticipated in the design, together with the repairs that have been made and the effect on the repairs of the action of high-velocity water.

In the author's absence, the paper was presented by Walter H. Price, U. S. Bureau of Reclamation, Denver, Colo.

## Research Open Session

The annual open research session Wednesday morning, under the direction of Stephen J. Chamberlin, Chairman, and George W. Washa, Secretary, produced papers on the plastic flow of reinforced concrete slabs and beams, the measurement of consistency of concrete by ball penetration, a new technique for determining stresses in steel reinforcement as a result of volume changes, instruments for measuring internal strain in concrete, the measurement of strain within a concrete mass, studies of concrete subjected to changes in temperature, the influence of thermal properties of aggregates on durability of concrete, field experiences with the soniscope in inspecting concrete, diagonal tension in reinforced concrete beams, the strength of concrete under triaxial loading, and tests of eccentrically loaded columns.

Those participating in the program included Paul G. Fluck, University of Wisconsin; J. W. Kelly, University of California; A. C. Loewer, Jr., Lehigh University; Rudolph C. Valore, Jr., National Bureau of Standards; E. C. Thoma, Purdue University; Delmar L. Bloem, National Sand and Gravel Assn.; Thomas B. Kennedy, Waterways Experiment Station; William J. Cheesman, Hydro-Electric Power Commission of Ontario; Arthur P. Clark, National Bureau of Standards; Douglas McHenry, U. S. Bureau of Reclamation; and Eivind Hognestad, University of Illinois.

## Questions Fired at Experts

Questions pertaining to compression tests, cracking, form stripping, tilt-up construction, curing, deterioration of concrete, and other problems were fired at a corps of experts during a panel discussion featured at the closing session of the convention Wednesday afternoon.

Discussion, from the audience as well as the panel, was monitored by Charles H. Scholer, Kansas State College, Manhattan, Kan. The board of experts consisted of A. Amirikian, Bureau of Yards and Docks, Navy Dept., Washington, D. C.; Miles N. Clair, Thompson and Lichtner Co., Inc., Boston; Raymond E. Davis, University of California, Berkeley; Bryant Mather, Concrete Research Division, U. S. Waterways Experiment Station, Clinton, Miss.; Douglas McHenry, U. S. Bureau of Reclamation, Denver; W. T. Moran, U. S. Bureau of Reclamation; Nathan M. Newmark, University of Illinois, Urbana; T. C. Powers, Portland Cement Assn., Chicago; Walter H. Price, U. S. Bureau of Reclamation; R. W. Spencer, San Marino, Calif.; and A. G. Timms, Bureau of Public Roads, Washington.

**The compression test cylinder—A weak reed?**

Standard cylinder compression tests for strength were strongly criticized by Charles E. Wuerpel, Marquette Cement Mfg. Co., Chicago. Declaring that the test was a "weak reed to lean on for control," Mr. Wuerpel asserted that cylinders are often cast or protected poorly, giving a false indication of the concrete in the structure; "the only way to obtain adequate control is to control all elements, including cement and aggregate manufacture, handling of materials, mixing and placing."

Mr. Wuerpel continued that findings from 28-day tests can be of little value since the concrete is already a permanent

part of the structure. He added that cylinders are cured under standard requirements which may not reflect curing conditions at the job site.

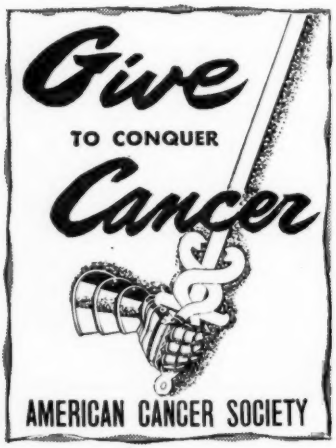
Others differed sharply. Mr. Clair said that it was not necessary to wait for results of the 28-day test; that concrete strength can be fairly well judged by 3- and 7-day tests. Compression test data can give an engineer an indication of the quality of the pour and permit him to catch poor work if it exists, he maintained.

### Find a better yardstick

The control cylinder strength is an indicated strength and there are many factors that influence it. "Find a better substitute," said Mr. Price, "and when that is done, we can discard the 6 x 12 control cylinder; it is still the best yardstick available."

### Modern procedures cause cracking

Modern construction practices are the cause of many of the problems associated with cracking, stated Professor Davis. "In more recent years, we have been bound to the goal of high-strength, fast-setting, fast-drying concrete." He said that less cracking was found in the low-cement concrete which gains strength slowly.




### Can't escape cracking

Mr. Clair said he had reached the conclusion that cracking cannot be averted. He said that the best approach to the problem is to follow good design, good construction and add to it dummy cracks at a spacing where cracks might occur. That way, he claimed, cracks will appear where wanted and will not be unsightly.

There is reason to believe that use of new deformed reinforcing bars will prevent wide cracks, said Professor Newmark. While the total amount of cracking with the deformed bars will be the same as in other types of construction, he believed there would be fewer large cracks.

### Curing effects cracking

Cracking can be prevented in highway work by withholding application of water to the concrete while it is rising in temperature and then throwing it on when the top temperature has been reached asserted M. A. Swayze, Lone Star Cement Corp., New York. The result, he explained, will be an expansion in the concrete due to absorption of water counteracting shrinkage due to loss of heat.

"I think the best answer that has been given to cracks is that you should never try to explain a crack because so many things are involved," observed Mr. Price.

### Ask the weather bureau

To avoid the danger of cracking and shock to concrete by form removal in winter, Mr. Spencer told the group that the contractor should look to the weather bureau for forecasts. Since forms should not be stripped if an extremely cold period is in the offing, forecasts of the bureau can be of invaluable assistance.

### Session a success

The audience participated wholeheartedly in the discussion and the large attendance, from the opening to final question, showed a keen interest in all the problems discussed. The session made it evident that there is still much to be learned about concrete and concrete construction.

# THANK YOU*

It is the privilege and pleasure of the American Concrete Institute to honor those who have done honor to the Institute while adding to their own stature and personal distinction in giving generously of their time, thought, energy and leadership in the cause of our objectives.

It is fitting that the Board of Direction, intrusted as it is with Institute management, should pause on such signal occasions as this the 46th annual convention to note factors contributing to the effectiveness of the structure of that management. The Board, while a continuing body, is constantly renewed from the membership. Each year some new officers and directors come to sit in the chairs of those ending their terms of official service. These usually continue as workers in the ranks for the same engrossing cause. In its attitudes and policies each year's Board bears distinctive marks of those who have gone before—the marks men made in the joint action of administration.

Thus a pause to express appreciation of and thanks for the services of those whose labors for the Institute in administrative and in technical accomplishments form the living tissue of the greater Institute of 1950, nurtured by the mutual effort through forty-five years.

In the spirit of this expression a debt is especially acknowledged in this year 1950 for the labors of all those who have played a variety of important parts in the efforts of the last five important years: in doubling ACI's membership; in envisioning new goals, achieving new lines of activity in an expanding policy; in an ever developing knowledge of things concrete; a doubled staff in new and efficient quarters; an enormously increased dissemination of that knowledge which is our primary aim.

Names of that half decade—Parsons, Gonnerman, Walker, Blanks, Gilkey—do not stop with a procession of able and perspicacious presidents, but include numerous directors in their less conspicuous but also able part, noting with special thanks those who, last year and this year, ended terms for which they were elected—Copeland, Kennedy, Klein, Norton and Washa—and Boase who did not survive his term.

Special appreciation is recorded of those called upon this year to make the current technical program outstanding in appeal; and of those also in charge of local arrangements for the general success of the occasion—Harry Delzell and the members of his committee: Frank W. Edwards, Harrison F. Gonnerman, William T. F. Hooper, Jr., John F. Seifried, C. P. Siess, Harry F. Thomson and W. W. Wallace; and for the all-important meeting facilities provided by Edgewater Beach Hotel and its capable and energetic staff.

*A resolution adopted by ACI Board of Direction, February 22, 1950.

## Important Changes Proposed for ACI Building Code

Major changes in reinforced concrete design and construction were advanced at a meeting of Committee 318, Standard Building Code, at the recent convention.

In a majority adoption of important new provisions for bond and shear as intended revisions of "Building Code Requirements for Reinforced Concrete" (ACI 318-47) the initial committee action starts ACI machinery toward the new code. Of the 28 members of Committee 318, 20 had returned approving ballots as this JOURNAL goes to press.

Committee 318 action however is not enough to change an ACI Standard—which is not a hasty procedure. When proposing a standard the committee's report goes to the ACI Standards Committee, responsible for its critical examination. If, when and as approved by the Standards Committee, a proposed new or revised standard then goes to the Technical Activities Committee for preliminary distribution (usually in the JOURNAL) as a basis for criticism and discussion prior to its formal convention presentation.

When a proposed revision is formally released by the Standards Committee, its originating committee may present it for adoption by an Institute convention after a specified interval (at least 30 days) following publication of the report. If approved by a majority of members present at a convention, it then goes to secret letter ballot of the Institute membership, as a whole, for publication of the new standard as such, requiring a ballot interval of 90 days. A new proposed standard becomes effective if ratified by two thirds of those voting in that ballot. The revisions proposed by Committee 318 have not as yet reached the early stage of first going to the Standards Committee (see Standards Committee, ACI Directory 1949, p. 26).

The proposed revisions are the result of extensive research on the bond between concrete and reinforcing bars. The 5-year program at the National Bureau of Standards (ACI JOURNAL, Dec. 1946, p. 383 and Nov. 1949, p. 161) is only a part of the research on this problem. The research resulted in the first standard for deformations on bars, "Standard Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement" (A.S.T.M. A 305-49). It is reported that within the next 2 or 3 months all mills will be rolling only bars which meet these specifications.

It is predicted that the use of the new bars with the changes in design prescribed by these proposed revisions in the code will result in better and more economical construction.

ACI Committee 208, Bond Stress, recommended, as the result of a study of all available data on bond, a completely new set of allowable unit stresses for bond. These were studied by a subcommittee of 318 and recommended to Committee 318 which has now approved them. With the unanimity of opinion so far shown in regard to the new stresses, there seems little doubt of their approval by the ACI Standards Committee in time for convention action February 1951, as a revision of the ACI Code 318-47. However, the ACI procedure on standards is such that this cannot be accomplished through final letter ballot of ACI Members until the middle of 1951.

Briefly, the changes decrease the allowable bond stress in plain bars (including the old types of deformed bars) and increase the allowable bond stresses for the new types of bars over those previously allowed for the old types. Top bars, those having more than 12 in. of concrete under them, are assigned lower bond stresses than bars in other positions. All plain bars must be hooked, which corresponds to special anchorage under the old and still existing provisions. The new bars develop sufficient anchorage by bond alone to correspond to special anchorage with the old type bars. Consequently, all bars under the new provisions correspond to those with special anchorage under the old provisions and code changes are proposed accordingly.

The changes approved by Committee 318, but not yet in the hands of the Standards Committee are as follows:

**104—Definitions**

Deformed bar. Delete first sentence and substitute:

A reinforcing bar conforming to the "Standard Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement" (A.S.T.M. Designation: A 305). Bars not conforming to these specifications shall be classed as plain bars.

Third line change "twelve" to "six".

**105—A.S.T.M. specifications cited in this code**

After A 185-37 add:

A 305-49 Standard Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement.

**207—Metal reinforcement**

Add a new subsection after subsection (*a*):

Deformations on deformed bars shall conform to the requirements of the "Standard Specifications for Minimum Requirements for the Deformation of Deformed Steel Bars for Concrete Reinforcement" (A.S.T.M. Designation: A 305).

**305—Allowable unit stresses in concrete**

In Table 305(a), delete first four items under "Shear" and all items under "Bond" and substitute attached Table 305(a).

**TABLE 305(a)—ALLOWABLE UNIT STRESS IN CONCRETE**

| Description | For any strength of concrete | Maximum value, psi | $f'_c$ 2000 | 2500 | 3000 | 3750 |
|---|---|---|---|---|---|---|
| **Shear:** $v$ (as a measure of diagonal tension) | | | | | | |
| Beams with no web reinforcement | $0.03f'_c$ | | 60 | 75 | 90 | 113 |
| Beams with properly designed web reinforcement | $0.12f'_c$ | | 240 | 300 | 360 | 450 |
| **Bond:** $u$ | | | | | | |
| **Deformed bars** (Except top bars) | | | | | | |
| Straight or hooked | | | | | | |
| In beams, slabs, one-way footings and as stirrups | $0.10f'_c$ | 350 | 200 | 250 | 300 | 350 |
| In two-way footings | $0.08f'_c$ | 280 | 160 | 200 | 240 | 280 |
| **Plain bars** (Except top bars) | | | | | | |
| In beams, slabs, one-way footings and as stirrups (Must have hooked ends) | $0.045f'_c$ | 158 | 90 | 113 | 135 | 158 |
| In two-way footings (Must have hooked ends) | $0.036f'_c$ | 126 | 72 | 90 | 108 | 126 |
| **Top bars** | | | | | | |
| Bars near the top of beams and girders having more than 12 in. of concrete under the bars | | | | | | |
| *Deformed bars* | $0.07f'_c$ | 245 | 140 | 175 | 210 | 245 |
| *Plain bars* (Must have hooked ends) | $0.03f'_c$ | 105 | 60 | 75 | 90 | 105 |

**505—Placing reinforcement**

(*a*) Replace second and third sentences with:

The minimum clear distance between parallel bars, except in columns, shall be equal to the nominal diameter of the bars.

**902—Ordinary anchorage requirements**

Delete "Ordinary" in title.

(*a*) Replace third sentence with the following:

The maximum tension in any bar must be developed by bond on a sufficient straight or bent embedment or by other anchorage (see Section 906).

(*b*) Line three delete "ten or more bar dimaeters" and substitute "six in.".

Delete second sentence.

(*c*) Replace this paragraph with:

In simple beams, or at the freely supported end of continuous beams, at least one-third the required positive reinforcement shall extend along the same face of the beam into the support a distance of six in.

**903—Special anchorage requirements**

Delete and substitute:

**903—Plain bars in tension**

All plain bars in tension shall terminate in standard hooks except that hooks shall not be required on the positive reinforcement at interior supports of continuous members.

**904—Anchorage of web reinforcement**

In (*a*)3, (*e*)3 and (*e*)4, change "$.04f'_c$" to "$0.045f'_c$" and "$.05f'_c$" to "$0.10f'_c$".

**905—Anchorage of bars in footing slabs**

(*a*) Line one, after "All" insert "plain".

Line two, second sentence after "hooks" insert "and the ends of deformed bars".

**1103—Spirally reinforced columns**

(*c*)1. Replace item 1 with the following:

For deformed bars with concrete having a strength of 3000 psi or above, twenty diameters of bar of intermediate or hard grade steel. For bars of higher yield point, the amount of lap shall be increased one diameter for each 1000 psi above 20,000 psi working stress. When the concrete strengths are less than 3000 psi, the amount of lap shall be one-third greater than the values given above.

(*c*)2. Change "twenty-five" to "100".

(*c*)3. Line three, change "1¼" to "1½".

**Note: These revisions are proposed only by their originating committee, they have not, as yet, replaced "Building Code Requirements for Reinforced Concrete" (ACI 318-47).**

## COMMITTEES MAP FUTURE ACTION

Chairmen of 16 committees reported actions taken in meetings held during the convention at Edgewater Beach Hotel, Chicago. In general, the attendance at committee meetings was good and, although not announced as open meetings, a number of visitors were present at most sessions.

**Committee 115—Research (Executive Group)**

Prof. Stephen J. Chamberlin, chairman, called a meeting of the Executive Group to discuss details for increasing the efficiency of the committee's work.

**Committee 208—Bond Stress**

Recommendations of Committee 208 on new bond stresses as summarized on pages 14 and 15 of the *News Letter* section of the April, 1949, JOURNAL, are now receiving attention by Committee 318, Standard Building Code, whence Committee 318 recommendations, if any, will be routed via the Standards Committee and the Technical Activities Committee for action by the Institute. With the new types of bars already actively on the market, it is not surprising that there should be a degree of impatience in some quarters with the Institute's version of an orderly procedure. At the recent meeting of Committee 208 a request was made by one of the visitors present that Committee 208 bestow upon the new-type bars, and its recommendation regarding them, "its quotable blessing." This, the members of Committee 208 present felt could properly be done as a re-affirmation of faith in its recommendation of a year ago.*

*It must be clearly understood that this "quotable blessing" is strictly a Committee 208 affair; that Committee 208 has no power to commit Committee 318, the Standards Committee, TAC or the Institute to any action, by implication or otherwise. Committee 318 will deal with the Committee 208 recommendation in whatever way it sees fit and the other bodies through which it must pass will do likewise.

There was some discussion of the numbering system for bar sizes recently proposed by manufacturers, since the terms "round," "square" and "diameter" have little descriptive significance for most of the new-type deformed bars. It was the consensus that the new numbering system represents a commendable simplification.

Current research developments on bond tests at the National Bureau of Standards were reported briefly as were some tests under way at the University of Texas investigating bond in the vicinity of cracks.

Committee members present included Herbert J. Gilkey, chairman, Arthur P. Clark, Chester L. Post, Raymond C. Reese and A. C. Weber.

In addition, there were 12 visitors: W. E. Bradbury, Frank B. Brown, William L. Clark, Jr., H. C. Delzell, Phil M. Ferguson, Orlando W. Irwin, G. F. Long, Nathan M. Newmark, Allen M. Reese, E. E. Rippstein, J. P. Thompson and C. A. Willson.

### Committee 210—Resistance to Erosion in Hydraulic Structures

Considerable progress was reported toward knowledge of factors influencing the erosion resistance of concrete. It was pointed out that the committee had been instrumental in having two papers* submitted to ACI on this subject during the year and that other information is being collected by Prof. Georg Wastlund in Europe and W. T. McClenahan in this country.

The committee did not feel that it had enough information to draft a report. It lacks information on leaching of concrete and the attack on it in sewers and around industrial plants.

It was observed that a report of the committee should cover all aspects of erosion in hydraulic structures, including abrasion, cavitation, leaching and chemical attack from sewage and industrial waste. The committee felt it should point out the properties of concrete that will aid resistance to erosion and also the limitations of concrete in such a capacity.

Committee members present were Walter H. Price, chairman, Jacob J. Creskoff and Roderick B. Young. Visitors included Frank H. Jackson, Emil Schmid and Nicolaas T. F. Stadtfeld.

### Committee 214—Evaluation of Results of Compression Tests of Field Concrete

The merits of the compression test as a measure of concrete quality were discussed at length by committee members and visitors. Data from compressive tests made by the Bureau of Reclamation and the Hydro-Electric Power Commission of Ontario were discussed. The committee wants more information from a variety of types of jobs before an evaluation.

Committee members included H. C. Ross, chairman, L. A. Dahl, Horace A. Pratt, Walter H. Price (proxy for W. A. Cordon), Stanton Walker, Charles S. Whitney and C. E. Wuerpel. Delmar L. Bloem, E. L. Hartman, and Roderick B. Young were visitors.

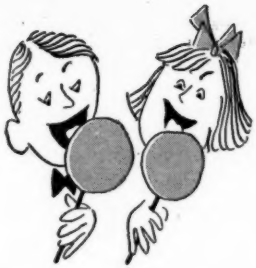
### Committee 314—Rigid Frame Bridges

The model test program for the skewed frame described by Milton Brumer in "Comparative Designs of a Segmental Skewed Frame Concrete Bridge by the Straight Line and Plastic Theory Methods," ACI JOURNAL, January, 1949, prompted considerable discussion. The test program will be conducted at the Virginia Polytechnic Institute under the direction of Chairman Dan H. Pletta.

With respect to an analysis of the skewed frame by relaxation, the committee reported that it looked feasible, but no actual work has been started.

It was also noted that the committee hopes to use a completely equipped oscillograph mobile unit in cooperation with the Institute of Transportation and Traffic Engineering, University of California. The unit, available for measuring

*"Resistance of Concrete and Protective Coatings to Forces of Cavitation," ACI JOURNAL, Oct. 1949, *Proc.* V. 46, p. 109 and "Effects of High-Velocity Water on Bonneville Dam Concrete," tentatively scheduled for June JOURNAL.



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pavement stresses, is ideally suited for measuring bridge impact stresses.

The chairman reported that the committee, growing in strength, should soon begin to have some worthwhile data available on the behavior of rigid frame bridges.

Members present were Professor Pletta, Douglas McHenry and Alfred L. Parme. H. R. Goodell, George E. Large and E. L. Reiter were guests.

### Committee 315—Detailing Reinforced Concrete Structures

The chairman reported that those corrections and revisions which had been developed by the committee had been submitted to the Standards Committee.

The committee also discussed the new-style deformed reinforcing bars and diagonal tension values. When, as and if approved, this would eliminate many hooks, shorten the laps and splices, and change details of beam steel at columns and supports.

Revisions were suggested for the detailing of hooks, especially right-angled bends, and the methods of dimensioning and scheduling. Discussion included the amount of reinforcing steel to be used in basement walls buried below ground, the proper hooks on stirrups and whether stirrups should be placed in an inverted position at the ends of continuous beams or possibly be changed to closed ties.

Comments and suggestions through the mails were read and discussed.

Members present included Raymond C. Reese, chairman, Frank H. Beinhauer, E. E. Ebling, Frank Kerekes, Henry L. Neve, E. E. Rippstein (by proxy) and John F. Seifried.

### Committee 321—Design of Reinforced Concrete Slabs

A proposed design specification for two-way floor slabs was discussed by the committee. The specification was presented at the Tuesday morning general session in the paper by Nathan M. Newmark and Chester P. Siess.

It was observed during the meeting that too often the ACI Building Code, and also state and municipal codes, tell the engineer *how* to design a structure instead of giving the limitations for design and leaving the choice of method up to the engineer.

Professor Newmark, chairman, V. P. Jensen and Professor Siess were committee members present and visitors included L. H. Corning and Maurice P. Van Buren.

### Committee 323—Prestressed Reinforced Concrete

The attendance at the meeting, six committee members and 21 visitors, was indicative of the great interest in the subject of prestressed concrete. Committee members included A. E. Cummings, chairman, A. Amirikian, Curzon Dobell, Nathan M. Newmark, Herman Schorer and H. B. Zackrison.

The question of specifications and codes for prestressed concrete was discussed at length but it was generally agreed that the entire subject is now in such a state of flux that it would be unwise to attempt to write definite specifications or a code of recommended practice.

Subcommittee IV, which is concerned with notations, definitions, specifications and codes, has started its work on notations and has assembled sets of symbols now used in various countries. These are being tabulated and it is expected that the committee will have a set of recommended symbols for prestressed concrete notation later this year.

Other subcommittees are at work assembling bibliographies on prestressing, studying patents, and collecting information on research projects being conducted in the United States and other countries.

### Committee 324—Precast Reinforced Concrete Structures

The committee considered the outline and partial draft of a proposed code applicable to precast concrete construction. It also discussed two papers, presented at the general session Tuesday morning by Chairman A. Amirikian, dealing with some of the important parts of the code, namely, minimum bar spacing, protective cover and extent and acceptability of cracked sections.

It was emphasized that the proposed acceptability clauses for cracking in precast concrete members should be part of every job specification and that it would be desirable to include them also in the ACI code.

Efforts will be made to complete the final draft of a standard for submission at the next ACI convention in San Francisco. About 20 members and visitors were present.

### Committee 325—Structural Design of Concrete Pavements for Highways and Airports

Members present included L. W. Teller, chairman, Henry Aaron, C. W. Allen (representing P. E. Masheter), Leo M. Arms, E. A. Finney, Bengt Friberg, Kenneth B. Woods and F. N. Wray. The program of the committee and work assignments were discussed.

### Committee 611—Inspection of Concrete

A brief informal meeting was attended by J. W. Kelly, chairman, William T. Neelands, T. C. Powers, R. W. Spencer, I. L. Tyler and Roderick B. Young.

Plans for revision of the *ACI Manual of Concrete Inspection* were discussed and responsibility for certain chapters assigned to individual members.

The committee hopes to complete the revision in the summer of 1950.

### Committee 612—Recommended Practice for Curing Concrete

The principal subject of discussion was the report of the committee which is to be resubmitted in a revised form within a few months.

A considerable portion of the meeting was also spent discussing a proposal made to Committee 612 that it sponsor a research project at the U. S. Bureau of Standards on petrographic and chemical studies of the effect of various curing procedures on the fundamental properties of hydrated cements in concrete. This project would be conducted at the Bureau under a research fellowship supported by the Calcium Chloride Assn. for research in concrete. The proposal was referred to the Technical Activities Committee which gave its approval.

Mark Morris, chairman, H. F. Clemmer, Frank H. Jackson, F. V. Reagel (by proxy), A. G. Timms and Paul M. Woodworth were present.

### Committee 613—Recommended Practice for the Design of Concrete Mixes

Subcommittee reports were presented on the effect of entrained air on concrete mixes, recommended percentages of air for air-entraining concrete, and mix design based on the Talbot method. These reports were discussed in some detail as they might affect the present standard.

Committee members present included Walter H. Price, chairman, Clayton L. Davis, A. T. Goldbeck, Fred Hubbard (proxy H. T. Williams), Frank H. Jackson, Henry L. Kennedy, I. L. Tyler and Stanton Walker. Visitors included Delmar L. Bloem, J. E. Gray, Thomas M. Kelly, and A. G. Timms.

### Committee 621 — Aggregate Selection, Preparation, Handling and Use

Those portions of the committee's assignment covering aggregate selection and preparation were discussed and work outlined for future meetings.

The following members of the committee were present: I. L. Tyler, chairman; A. T. Goldbeck; Bror Nordberg; R. W. Spencer; A. G. Timms; Stanton Walker; William R. Waugh (proxy for Byram W. Steele); and H. T. Williams (proxy for Fred Hubbard).

### Committee 711—Precast Floor Systems for Houses

The discussion was directed toward the justification of, and ways and means whereby, modifications in the ACI Building Code covering spacing of reinforcing in precast concrete units could be effected.

Committee members and visitors included F. N. Menefee, chairman, A. Amirikian, Stephen J. Chamberlin, George E. Large, E. F. Massie, C. F. Moore, Dean Peabody, Jr., Dan H. Pletta, H. S. Robins, Herman Schorer and George W. Washa.

# Who's Who This Month

## Herbert J. Gilkey

As retiring president of the Institute, Herbert J. Gilkey, Head of the Dept. of Theoretical and Applied Mechanics, Iowa State College, Ames, Ia., addressed the 46th annual convention delegates at the noon luncheon on "The Zig-Zag Course of Concrete Progress," p. 573.

Professor Gilkey has long been active in Institute affairs, both on technical and administrative committees. He is chairman of the Standards Committee and Committee 208, Bond Stress, as well as a member of the ACI Committee 115, Research. He is ACI's representative on the Highway Research Board, and is active in A.S.C.E., A.S.T.M., and S.P.E.E.

Along with S. J. Chamberlin and R. W. Beal, he was Wason Medalist for the most meritorious paper for their 1939 paper "The Bond Between Concrete and Steel." Professor Gilkey was a member of the Publications Committee, 1939-46; Board of Direction, 1937-38 and 1945-46; and vice-president, 1947-48. He is the author of five JOURNAL papers.

## A. L. Malcolm and R. B. Young

Unusual construction methods applied to mass concrete are described in "Concreting on the Ottawa River Projects of the Hydro-Electric Power Commission of Ontario," p. 581, by A. L. Malcolm and R. B. Young, Assistant Engineer, Engineering Division, and Assistant Director of Research, respectively, Hydro-Electric Power Commission of Ontario, Toronto, Ont., Canada.

Mr. Malcolm has been with the Commission since 1917 as superintendent and field engineer on water power developments throughout the Province of Ontario, more recently in the latter capacity at the Des Joachims project. He graduated from the University of Toronto in 1911; from 1911-13 he was employed by Smith, Kerry and Chace, consulting engineers, Toronto, on water power development design and construction.

He also served as an electrical engineer with the Riordan Paper Co., 1913-14; the Water Power Branch, Dept. of the Interior, Ottawa, on the Lake of the Woods level investigation, 1914-16; and was superintendent of construction for the Golden Lake Dam and dredging, 1916-17.

Mr. Young is well-known to JOURNAL readers as a past president of the Institute and author of numerous papers, as well as being active on administrative and technical committees. He also has the enviable record of having attended every ACI convention since 1917.

He received the Wason Medal for "the most meritorious paper" for his 1937 paper "Concrete—Its Maintenance and Repair."

The research division of the Commission, of which Mr. Young is Assistant Director, is conducting applied research in many fields of science, chemistry, electricity, metals, mechanics and structures, including the inspection of concrete and soils structures.

To quote Mr. Young: "Although others may not agree with me, I consider my most important contribution in the field of concrete to be the development of field methods for controlling quality of concrete based on Abram's water-cement ratio theory; here it is interesting to note that the Commission was the first organization to apply the water-cement ratio in a practical way, having used it in 1919 on a small power development and later in 1920-21 on two larger jobs, one of which was the Queenston power development on the Niagara River which contained upwards of 600,000 cu yd of concrete.

"The total quantity of concrete that has been placed by the Commission would, by this time, total three or four million cu yd."

*Continued on p. 30*

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Mr. Young has been actively interested in the studies of the durability of concrete in service as well as other fields including concrete control and inspection, winter concreting, construction, and maintenance and repair methods.

He was recently appointed chairman of the Committee on Concrete and Reinforced Concrete of the Canadian Standards Assn., which in its scope duplicates in a large part that of the Joint Committee of the same title. He was also one of the original organizers of the National Ready Mix Concrete Assn. and is now an honorary member of the organization.

## N. M. Newmark and C. P. Siess

are not newcomers to JOURNAL pages; both are on the staff of the University of Illinois as Research Professor of Structural Engineering and Research Assistant Professor of Civil Engineering, respectively. Their latest contribution, "Proposed Design Specifications for Two-Way Floor Slabs," p. 597, was presented at the 46th annual convention in Chicago.

At the recent convention, both men received the Wason Medal for "the most meritorious paper" of the year for "Rational Analysis and Design of Two-Way Concrete Slabs" which appeared in the December, 1948, JOURNAL.

Doctor Newmark is also active in ACI work as a director, as member of the Technical Activities Committee and as chairman of Committee 321, Design of Reinforced Concrete Slabs.

He was graduated with special honors in civil engineering at Rutgers University in 1930, and then went to the University of Illinois as a research graduate assistant in civil engineering. Since then he has been continuously on the university staff, and was appointed to his present position in 1947.

In addition to his ACI affiliation, he is a member of A.S.C.E., A.S.M.E., Institute of Aeronautical Sciences and International Assn. of Bridge and Structural Engineers. He is also a member of A.S.T.M., A.R.E.A., Society of Experimental Stress Analysis, Phi Beta Kappa, Tau Beta Pi, Sigma Xi and Chi Epsilon.

Professor Siess graduated in civil engineering at Louisiana State University and then worked as soils engineer in the testing laboratory of the Louisiana Highway Commission. In 1937 he became special research graduate assistant in theoretical and applied mechanics at the University of Illinois where he received his M. S. in 1939 and his Ph.D. in 1948.

After work with the Chicago Dept. of Subways and Superhighways and the New York Central Railroad he returned to the University of Illinois in 1941 as special research associate and special research assistant professor of theoretical and applied mechanics, and was appointed to his present position in 1949.

Chairman of ACI Committee 215, Fatigue of Concrete, and member of Committee 321, Design of Reinforced Concrete Slabs, and Committee 115, Research, Professor Siess also is affiliated with A.S.T.M., Highway Research Board, A.S.C.E., S.E.S.A., Sigma Xi, Tau Beta Pi, Phi Kappa Phi, Omicron Delta Kappa and Phi Eta Sigma.

## Leonard E. Dunlap

"Inspection of Building Construction," p. 609, by Leonard E. Dunlap, president of Carr and Wright, Inc., architects and engineers, Chicago, Ill., was one of the papers briefed at the inspection session of the 46th annual convention.

Mr. Dunlap graduated from the University of Illinois in 1917 in architectural engineering and worked as a structural designer for Swift and Co., Illinois Central Railroad, Sinclair Oil Co., and Nimmons, Carr and Wright, architects from 1917 to 1920. He served as district engineer for Kalman Steel Co., 1920-26 and chief engineer, for Nimmons, Carr and Wright, 1926-42.

From 1942 to 1944, Mr. Dunlap served as consulting engineer for Charles T. Main, Inc., architects and engineers, and returned to the firm of Nimmons, Carr and Wright in 1944. In 1947 the firm was incorporated under the name of Carr

and Wright, Inc., and Mr. Dunlap became president of the firm in 1948, in charge of structural design, field supervision and business administration.

## A. R. Alderman, A. J. Gaskin, R. H. Jones and H. E. Vivian

Members of the Division of Industrial Chemistry of the Commonwealth Scientific and Industrial Research Organization, Melbourne, Australia, A. R. Alderman, A. J. Gaskin, R. H. Jones and H. E. Vivian are newcomers to the JOURNAL pages, with "Australian Aggregates and Cements in Relation to Cement-Aggregate Reaction," p. 613.

Dr. Alderman, Senior Principal Research Officer, helped establish the Section of Cement and Ceramics for the Commonwealth Scientific and Industrial Research Organization. He graduated from the University of Adelaide in 1925, receiving his Ph. D. at Cambridge and D. Sc. at Adelaide. Apart from two years spent in England at Cambridge University, Dr. Alderman was a lecturer and researcher at the University of Adelaide until 1942 when he joined the Division of Industrial Chemistry.

Mr. Jones, graduating from the University of Adelaide in 1940, enlisted in the Royal Australian Air Force but was transferred to the Dept. of Munitions in 1942. He joined the Division of Industrial Chemistry in 1944 and has been concerned mainly with the petrology of cement clinkers and aggregates.

Biographical sketches of Messrs. Gaskin and Vivian appear below.

## H. E. Vivian

"Some Australian Studies on Cement-Aggregate Reaction in Mortar," p. 617, by H. E. Vivian summarizes some of the work in Australia on cement-aggregate reaction.

He graduated in agricultural science at the University of Sydney in 1941. After spending some time as a chemist with the Standard Portland Cement Co. in New South Wales, he joined the Division of Industrial Chemistry in 1942.


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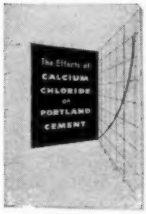


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His investigational work in the Section of Cement and Ceramics has been largely on the subject of cement-aggregate reactions.

## A. J. Gaskin

A newcomer to JOURNAL pages, A. J. Gaskin, Division of Industrial Chemistry of the Commonwealth Scientific and Industrial Research Organization, Melbourne, Australia, is the author of "Carbon Dioxide and the Cement-Aggregate Reaction," p. 625.

The author of several technical bulletins in Australia, Mr. Gaskin graduated in 1940 from the University of Melbourne, and received an M. S. in 1941. He joined the Division of Industrial Chemistry in 1942. Although he was appointed lecturer in geology at the University of Melbourne in 1945, he has retained his association with the Division's Section on Cement and Ceramics

In addition to general petrology, his research work has been mainly concerned with clay mineralogy and cement.

## F. N. Menefee and H. L. Kinnier

"Spacing of Moment Bars in Precast Joists," p. 629, by F. N. Menefee, Professor of Engineering Materials, University of Michigan, Ann Arbor, Mich., and H. L. Kinnier, Assistant Professor of Civil Engineering, University of Virginia, Charlottesville, Va., is one of two papers about precast units in this issue.

Professor Menefee is chairman of Committee 711, Precast Floor Systems for Houses, and is also active in A.S.C.E. For many years he had charge of the Materials Testing Laboratory at the University of Michigan and he is now teaching a graduate course in prestressed reinforced concrete. He also worked on many reinforced concrete structures while employed by the Fargo Engineering Co., Jackson, Mich.

Professor Kinnier was appointed to his present position in 1947; prior to that he had served as acting assistant professor at the University of Virginia and served with the Bridge Design Office of the Virginia Dept. of Highways. He received a B. S. in civil engineering from the University of Virginia in 1942 and immediately entered the U. S. Navy where his duties included ship repair superintendent and dry docking duties at the Norfolk Navy Yard and officer in charge of floating dry docks at an advance repair base in the Philippine Islands. He is also a member of A.S.C.E., A.S.E.E., Theta Tau and Phi Eta Sigma.

## Arsham Amirikian

As chairman of ACI Committee 324, Precast Reinforced Concrete Structures, Arsham Amirikian, Head Designing Engineer, Bureau of Yards and Docks, Navy Dept., Washington, D. C., has long been actively interested in precast concrete.

"Proposed Specifications for Minimum Bar Spacing and Protective Cover in Precast Concrete Framing Members," p. 637, is the latest of many articles written by Mr. Amirikian, three of which have appeared in the JOURNAL.

After receiving his C. E. degree from Cornell University in 1923, he spent five years with private engineering and fabricating firms. In 1928 he joined the Bureau of Yards and Docks of the Navy Dept. and has been with the Bureau for almost 22 years, beginning as an assistant structural engineer and being appointed to his present position in 1945.

For the past 12 years he has also headed the Bureau's special design section on buildings, floating structures, welding and protective- construction. In the latter capacity, he has been associated with most of the important structures built by the Bureau, including the Navy's huge floating drydocks and gate caissons built of reinforced concrete.

In recent years he has devoted most of his time to the technique of precast concrete construction. He developed the thin-shell "hollow-rib" and "cellular" precast concrete framing systems utilized in a number of Navy shore and floating structures. He is presently engaged in adapting these framing systems to protective construction.

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## A. T. Goldbeck

A. T. Goldbeck, Engineering Director of the National Crushed Stone Assn., is the author-chairman of "The Problem of Slab Dimensions," p. 641, presented at the annual convention by Subcommittee 2, Committee 325, Structural Design of Concrete Pavements for Highways and Airports.

Mr. Goldbeck was graduated from the University of Pennsylvania with a B.S. in civil engineering in 1906, and served as instructor at the university for two years and during the succeeding two years at Lafayette College.

He joined the staff of the Office of Public Roads in 1910, became Assistant Engineer in Charge of the Philadelphia Municipal Laboratory in 1913, rejoined the reorganized laboratory of the Bureau of Public Roads and Rural Engineering in 1915, and served successively as Engineer of Tests and Chief of the Division of Tests. In these capacities he conducted and directed researches in concrete pavement design which included studies of stress distribution under static and moving loads, subgrade pressure measurements, studies of subgrade friction, impact studies, the flow of concrete, measurements of expansion and contraction, and other studies pertaining to pavement design.

A newly elected vice-president of the Institute, Mr. Goldbeck has served on the Advisory Committee, 1922-23; Publications Committee, 1944-46; Board of Direction, 1948-50. In addition to being the author of seven ACI papers, he has served on many technical committees of ACI and other national organizations.

In addition to Committee 325, he is a member of Committee 115, Research; Committee 613, Recommended Practice for the Design of Concrete Mixes; 617, Specifications and Recommended Practice for Concrete Pavements and Bases; and 621, Aggregate Selection, Preparation, Handling and Use. He has been chairman of A.S.T.M. Committee C-9 on Concrete and Concrete Aggregates, chairman for many years of the Design Committee of the Highway Research Board, and member of two of the Joint Committees on Concrete and Reinforced Concrete.

Other members of Subcommittee 2 of Committee 325 include Leo M. Arms, Portland Cement Assn., Chicago; H. L. Lehmann, Louisiana Dept. of Highways, Baton Rouge, La.; P. E. Masheter, Ohio Dept. of Highways, Columbus, Ohio; L. A. Palmer, Bureau of Yards and Docks, Navy Dept., Washington, D. C.; and F. N. Wray, Highway Research Board, Washington, D. C.

## Eivind Hognestad and C. P. Siess

Another paper from the University of Illinois campus at Urbana, Ill., "Effect of Entrained Air on Bond Between Concrete and Reinforcing Steel," p. 649, is by Eivind Hognestad, Special Research Associate of Theoretical and Applied Mechanics, and C. P. Siess, whose short biographical sketch appears on p. 30.

Mr. Hognestad's first ACI paper appeared in the February 1950 issue, and he has earlier contributed to discussion, "Letters from Readers" and has reviewed many Scandanavian publications in the "Current Reviews" section of the JOURNAL. He is also the author of a paper on reinforced concrete footings published in Sweden and co-author of a Highway Research Board paper on bond.

In 1947 he received an appointment as a research graduate assistant at the University of Illinois where he worked on various projects pertaining to reinforced concrete research under the supervision of Prof. F. E. Richart and received his M.S. in 1949. He was appointed to his present position in 1949.

Mr. Hognestad graduated from Norway's Institute of Technology in 1947 as a civil engineer. Following a period as a political prisoner in 1942-43, he escaped to England in 1944 and entered the Royal Norwegian Navy, returning to the Institute in 1946. He is a member of ACI and the Norwegian Society of Engineers.

# Honor Roll

**February 1 to February 28, 1950**

**In this record of the first month of the new Honor Roll (Feb. 1, 1950 to Jan. 31, 1951) it should be recorded that, Newlin D. Morgan, Prof. of Architectural Engineering, University of Illinois, was at the Chicago convention. He insists there is no special magic in his staying at the top of the list; that his students and others seem to recognize that membership in ACI is a bread-and-butter factor of their chosen educational channels. Professor Morgan seems a modest chap; probably there is some magic in the way he presents concrete as a dominant factor in architectural engineering progress and that a lot of grade A thinking of a lot of grade A engineers pours through ACI pages and keeps the subject interesting. Still and all, ACI is grateful to Professor Morgan. More and more Student Members (at a low dues rate) who stick by the subject when they get out on the job seem to see that education is a continuous process for grade A jobs; many become Junior and Individual Members when they are thinking in a field where thinking does not congeal in a complacent world of text books that never seem to catch up with knowledge growth.**

**Newlin D. Morgan (Ill.).....34**
**R. H. Sherlock (Mich.).....3½**
**Emil A. Gramstorff (Mass.).....2**
**Henry E. Griset (N. C.).....2**
**Walter N. Handy (Ill.).....2**
**Wm. T. F. Hooper, Jr. (Ill.).....2**
**H. J. Gilkey (Ia.).....1½**
**Roderick B. Young (Canada).....1½**
Sterling Lowe Bugg (Fla.).....1
W. Fisher Cassie (England).....1
Jacob J. Creskoff (D. C.).....1
Harmer E. Davis (Calif.).....1
DeWitt Dieterich (Ill.).....1
Belmon U. Duvall (Ohio).....1
William E. Gabelman (Calif.).....1
H. F. Gonnerman (Ill.).....1
David V. Lewin (Ohio).....1
Harry E. Lewis (N. Y.).....1
Glenway Maxon (Wis.).....1
James A. McCarthy (Ind.).....1
H. J. McGillivray (Fla.).....1
William D. Nowlin (Va.).....1
Wm. Otcovsky (Ill.).....1
Stanley A. Phillip (Ill.).....1
Theodore O. Reyhner (Colo.).....1
Evan L. Richard (Australia).....1
Horace B. Rickey, Jr. (La.).....1
Emil Schmid (N. J.).....1
Pascal L. Tasio (N. Y.).....1
Wm. H. Thoman (Colo.).....1
H. F. Thomson (Ill.).....1
F. E. Votaw (Mass.).....1
R. A. Zern (Pa.).....1

The following credits are, in each instance "50-50" with another member.

Frederick G. Anderson
C. Merrill Barber
George E. Barnes
A. Allan Bates
J. G. Bragg
R. A. Burmeister
D. J. Cameron
Julian B. Carson
Aloysius E. Cooke
William A. Crabb
M. A. Craven
Harry Delzell
W. J. Emmons
A. T. Goldbeck
Homer M. Hadley
John M. Hayes
W. R. Ingram
Frank H. Jackson
Frank Kerekes
W. D. Kimmel
Ralph W. Kluge
Irvin H. Luke
Ian Macallen
Harmon S. Meissner
Henry Pfisterer
Leo F. Pratt
John A. Ruhling
Wilfrid Schnarr
John J. Stedje
G. E. Troxell
Paul M. Trueblood
Lewis H. Tuthill
Julius J. Warner
Willard W. Warzyn
Clement T. Wiskocil

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# New Members

**Of the 124 applications for membership approved for the month of February, 60 are Individual, 5 Corporation, 10 Junior and 49 Student.**

**Illinois was top membership state for the month providing 41 new Members. California, New York, Ohio and Wisconsin each provided 6 new applicants.**

**ACI's membership total February 28, 1950, became 5013.**

### California

Horonjeff, Robert, (Indiv.) Institute of Transportation & Traffic Engineering, University of California, Berkeley, Calif.

Lin, Tung Yen, (Indiv.) Division of Civil Engineering, University of California, Berkeley 4, Calif.

Niesley, Myron B., (Indiv.) 1429 Santa Fe Ave., Los Angeles 21, Calif.

Robinson, Parker M., (Indiv.) c/o Pittsburgh Testing Laboratory, 651 Howard St., San Francisco 5, Calif.

Stanovich, Frank A., (Indiv.) California Homes & Materials Co., 855 E. Meridian St., Duarte, Calif.

Thompson, H. "LV", (Indiv.) c/o Bureau of Power & Light, 504 S. Hill St., Room 515, Los Angeles 13, Calif.

### Colorado

Alvarez, Adolfo A., (St.) 2195 S. Franklin, Denver 10, Colo.

Crabb, Kenneth R., (Indiv.) 2501 S. University Blvd., Denver 10, Colo.

Sailer, Robert, (Indiv.) 1040 Niagara St., Denver 7, Colo.

Wahler, William A., (St.) M-3 Vetsville, Boulder, Colo.

### Connecticut

Loomis, Robert W., (Indiv.) 32 Maple Ave., Windsor, Conn.

### District of Columbia

Bloem, Delmar L., (Indiv.) National Ready Mixed Concrete Assn., 1325 E St., N.W., Washington 4, D. C.

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*Continued on p. 41*

# Positions and Projects — ACI Members

## Research compilation available

Committee 115, Research, released its annual compilation of research projects at the research session of the 46th Annual Convention in Chicago.

The annual compilation is in three parts: (1) lists all educational institutions that are conducting concrete research and lists the projects under consideration; (2) provides similar information for non-educational organizations; (3) classifies projects under subject headings.

This year, 52 organizations reported activities, including universities, highway departments, federal departments, cement companies, industrial laboratories, commercial laboratories and various associations.

The reporting organizations listed 315 different projects, covering phases of research in the field of aggregates, admixtures, cements, and plain and reinforced concrete.

Members may obtain a copy of the compilation (while they last) by writing to Institute headquarters.

## Wanted—Certain old ACI Directories

Checking through old ACI Directories of membership, Institute headquarters finds that copies for the years 1927, 1928, 1932, 1933 and 1935 are missing. We regret that we lack positive knowledge that membership lists were published in some of those years. Would any member who can advise where missing copies might be obtained please notify the Secretary, 18263 W. McNichols Rd., Detroit 19, Mich.

## ACI member appointed to water resources commission

Samuel B. Morris, Dept. of Water and Power, Los Angeles, Calif., has been appointed to membership on the temporary Water Resources Policy Commission by President Truman. The committee is to study and make recommendations on existing federal legislation and policies in the water resources field.

Though the commission will deal principally with the large-scale water problems of the West, the current water shortage in New York City is cited as the sort of problem that should be considered under the heading of federal participation in municipal water supply projects.

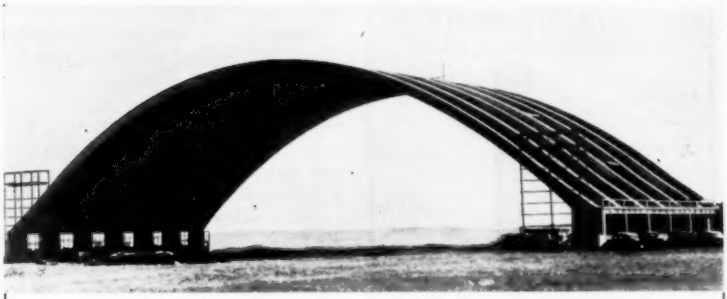
The Executive order establishing the commission states that it shall give consideration in particular to (1) the extent and character of federal government participation in major water resources programs, (2) an appraisal of the priority of water resources programs from the standpoint of economic and social need, (3) criteria and standards for evaluating the feasibility of such projects, and (4) desirable legislation or changes in existing legislation.

## Iowa State holds Better-Concrete Conference

A Better-Concrete Conference was sponsored by the Dept. of Theoretical and Applied Mechanics and Engineering Extension Service at Iowa State College, Ames, Ia., in March. The conference featured discussions on the practical procedures, techniques and problems of producing better concrete.

Herbert J. Gilkey, Professor and Head, Dept. of Theoretical and Applied Mechanics, Iowa State College, and past president of ACI, discussed the "Enigmas of Plain Concrete." H. K. Howell, Lieutenant-Colonel, Corps of Engineers, U. S. Army, and also an ACI Member, was the dinner speaker.

Other ACI Members who participated in the 3-day program included Stephen J. Chamberlin, Professor and Research Professor of Theoretical and Applied Mechanics, Iowa State College, and chairman of ACI Committee 115, Research; Frank Kerekes, Assistant Dean of Engineering and Professor of Structural Engineering,



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Iowa State College; William Lerch, Manager of Applied Research Section, Research and Development Laboratories, Portland Cement Assn., Chicago; Fred F. Loy, District Engineer, Portland Cement Assn., Des Moines, Ia.,; Maurice C. Miller, Vice-President, Hawkeye Portland Cement Co., Des Moines, Ia.; and Stanton Walker, Director of Engineering, National Ready Mixed Concrete Assn. and National Sand and Gravel Assn. Washington, D. C.

Papers were presented on concrete practice in highways, structures and farm adaptations. The problems and practices in ready-mixed concrete production were discussed. Other topics included durability of concrete, mixture design, researches in concrete, and the relation of the proposed Iowa Building Code to concrete construction.

## Couse and Maxon named A. G. C. officers

Walter L. Couse, president of Walter L. Couse and Co., Detroit, Mich., was named president of the Associated General Contractors of America at their 31st annual convention in San Francisco, Calif. The new vice-president is Glenway W. Maxon of the Maxon Construction Co., Dayton, Ohio.

Mr. Couse, who was A. G. C. vice-president during 1949, is perhaps best known for the work he has done in construction of industrial buildings for the automobile industry in Detroit. His activities have also included municipal and industrial construction in several states and considerable bridge construction. Mr. Maxon's firm is primarily known for its work on inland waterways, construction of bridges and navigational works, industrial facilities, subaqueous foundations and shipyards.

H. C. Turner, Jr., Turner Construction Co., New York City, was re-elected as a director of the building division and also will serve as the new vice-chairman of the Building Contractor's Division. C. L. Davis, Miller-Davis Co., Kalamazoo, Mich., was also named to a directorship in the building division.

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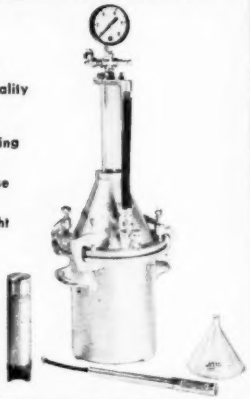

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## Pressure pipe group affiliates with American Concrete Pipe Assn.

At the recent concrete pipe industry convention held in San Francisco, Calif., the newly formed American Concrete Pressure Pipe Assn. affiliated with the 43-year old American Concrete Pipe Assn.

The two national concrete pipe associations will coordinate their activities as members of a joint coordinating and affiliating association known as Concrete Pipe Assn., Inc., with headquarters in Chicago and under the direction of H. F. Peckworth, Managing Director.

The newly named officers of the American Concrete Pipe Assn. are: president, Edward H. Fox, Cincinnati, Ohio; vice-presidents, Hugh P. Ford, Eugene, Ore., Harry Leuliette, Columbus, Ohio, George W. Black, Houston, Texas; and directors, H. W. Heath, East Orange, N. J., J. A. Dunn, Swampscott, Mass., and E. H. Eby, Helena, Mont.

New officers of the American Concrete Pressure Pipe Assn. include: president, Robert V. Edwards, Los Angeles, Calif.; vice-president, P. R. Hirsh, East Orange, N. J.; and secretary-treasurer, Harry S. Price, Jr., Dayton, Ohio.

## Behind the "Bamboo Curtain"

Arthur B. Morrill, Shanghai, China, and author of "Unusual Features in Design and Construction of a 20-Million Gallon Concrete Reservoir," October 1931 JOURNAL, wrote recently that business is bad in China because of the blockade, but except for that the change has not been as great or calamitous as many people feared. For a while the blockade was so tight that no mail got through from the United States for months, he said.

He also said that the political and military situation has been "pretty hot" around Shanghai.

"Since then we have passed behind the 'Bamboo Curtain'," he said, "which however, is not opaque."

## Pavement design trends

A report recently issued by the American Road Builders' Assn. Committee on Concrete Pavement Design presents the trends in concrete pavement practices of the 48 state highway departments and the District of Columbia. This 122-page booklet, A.R.B.A. *Technical Bulletin* No. 163, has been compiled with the assistance of representatives from each of the highway departments and covers subgrade soil practices, concrete materials, proportioning, placing, finishing and curing, as well as structural features of concrete pavements.

Members of the committee were A. T. Goldbeck, Engineering Director, National Crushed Stone Assn. and ACI vice-president, Washington, D. C.; T. J. Kauer, Director of Highways, Ohio Dept. of Highways, Columbus, Ohio; Tilton E. Shelburne, Director of Research, Highway Research Council, University of Virginia, Charlottesville, Va.; E. W. Bauman, Secretary, National Slag Assn., Washington, D. C.; and H. F. Clemmer, Engineer of Materials, District of Columbia.

## University of Michigan plans conference on concrete

Preliminary arrangements have been made for a conference on concrete at the University of Michigan, Ann Arbor, Mich., April 26 and 27. The program is being planned by a joint committee from the College of Architecture and Design, the Dept. of Civil Engineering and the Portland Cement Assn.

The general theme of the conference will be the proper use of concrete, placing special emphasis on construction. Topics on the program include forming, exposed concrete, prestressing and heavy duty floor construction.

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## New Members

*Continued from p. 37*

### Ohio

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Sykes, W. B., (Indiv.) Ubique the Drive, Pishiobury Park, Sawbridgeworth, Herts, England

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*BRIELMAIER, A. A., Washington University, St. Louis 5, Mo.
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*BROCK, A. S., c/o Frank Kornacker, 330 S. Wells St., Chicago, Ill.
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BRONOWSKI, THEODORE, Valparaiso University, Valparaiso, Ind.
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*BROWN, FRANK B., Wire Reinforcement Inst., Inc., 1049 National Press Bldg., Washington 4, D. C.
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BULLOCK, R. E., Washington University, St. Louis, Mo.
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*CAIN, CRAIG J., Combustion By-Products Co., 228 N. LaSalle St., Chicago, Ill.
CALIENDO, EDWARD J., Ill. Inst. Tech, Chicago, Ill.
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*CHAMBERLAIN, S. J., Iowa State College, Ames, Ia.
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**Price 50 cents (in special covers).**

REPORT of COMMITTEE 616—Sept. 1949, pp. 1-16 (V. 46)

**Supersedes 38-30 and 45-18.**

This ACI standard establishes recommended practices for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage, and general characteristics and factors affecting durability.

## AN ULTRASONIC METHOD OF STUDYING DETERIORATION AND CRACKING IN CONCRETE STRUCTURES ..... 46-2

**Price 35 cents.**

J. R. LESLIE and W. J. CHEESMAN—Sept. 1949, pp. 17-36 (V. 46)

A new method and apparatus for field and laboratory testing of concrete is described. The apparatus called the "Soniscope" was originally designed to detect internal cracks in concrete. It develops pulses of ultrasonic sound in the material and measures the velocity of their transmission through it. This pulse velocity has the unique advantage of being independent of the size or shape of the body under test. Measurements can be made with equal facility in mass concrete, slabs or laboratory specimens.

The existence and extent of internal cracks and the depth of visible surface cracks can be determined by the use of this apparatus.

The velocity has been found, by experiment, to be a reliable measure of the condition of the concrete and is particularly useful in deterioration studies. The dynamic modulus can be calculated from this velocity, and values so obtained are found to agree closely with the results of tests using established methods.

## MANUFACTURE OF REINFORCED FOAM CONCRETE ROOF SLABS ... 46-3

**Price 35 cents.**

I. T. KOUDRIASHOFF—Sept. 1949, pp. 37-48 (V. 46)

The Russian type of lightweight concrete described used a rosin-glue emulsion to preserve the air voids before the initial set of the cement. Shrinkage was decreased and strength increased through high pressure steam curing. The autoclave treated foam concrete used in the production of precast industrial roof slabs had a unit weight of 47 lb per cu ft and a compressive strength of over 500 psi. The lightweight slabs, used in a load carrying capacity and as insulation, reduced construction time by 50 percent and costs by as much as 20 percent. Test data on roof slabs and production procedures are also described.

## SUGGESTIONS ON CONCRETE FLOOR CONSTRUCTION ..... 46-4

**Price 35 cents.**

ERNST GRUENWALD—Sept. 1949, pp. 49-56 (V. 46)

The relationship between good concrete floors and the proper selection of cement and aggregates is discussed. Data are cited to emphasize the advantage of coarse-aggregate mixes over cement-sand topping for concrete floors.

## USE OF AIR-ENTRAINING CONCRETE IN CANAL LINING ..... 46-5

**Price 35 cents.**

JOSEPH J. WADDELL—Sept. 1949, pp. 57-64 (V. 46)

This paper presents a discussion based on field observations of the use of air entrainment in canal lining concrete. The Bureau of Reclamation made studies of air-entraining agents for use in the irrigation canal lining for the Friant-Kern Canal on the Bureau's Central Valley Project in California. Results to date indicate that appreciable benefits accrue when an air-entraining agent is used in concrete which is placed and compacted by means of a mechanical slip-form. Care is necessary in adjusting concrete mixes to incorporate entrained air because of the sensitivity to mix changes of concrete for slip-form placement.

## THE USE OF PORTLAND-POZZOLAN CEMENT BY THE BUREAU OF RECLAMATION .. 46-6

**Price 35 cents.**

ROBERT F. BLANKS—Oct. 1949, pp. 89-108 (V. 46)

The Bureau of Reclamation has made extensive studies of pozzolanic materials, and portland-pozzolan cements are now being used in many of the major structures built by the Bureau. The properties of portland-pozzolan cement that are used advantageously in the production of mass concrete are described.

## RESISTANCE OF CONCRETE AND PROTECTIVE COATINGS TO FORCES OF CAVITATION ..... 46-7

**Price 35 cents.**

WALTER H. PRICE and GEORGE B. WALLACE—Oct. 1949, pp. 109-120 (V. 46)

A machine used for producing cavitation erosion in the laboratory is described and the results of tests made to investigate the effect of mix proportions, surface treatment, and protective coatings on the resistance of concrete to cavitation are discussed. Through proper use of these, the resistance of concrete surfaces to cavitation erosion may be extended three or four times, but even the best concrete will not resist the forces of cavitation for a prolonged period. Heavy rubber coatings bonded well to the surface of the concrete have proved effective.

## VACUUM PROCESSES APPLIED TO PRECAST CONCRETE HOUSES ... 46-8

**Price 35 cents.**

K. P. BILLNER and BERT M. THORUD—Oct. 1949, pp. 121-128 (V. 46)

The use of vacuum processes in precast concrete construction simplifies the building of fire-resistant, durable structures designed to permit the maximum use of like units. Vacuum processes have thus far been used in one- and two-story houses and one-story industrial or warehouse structures, but further developments should make possible similar benefits for multi-story construction.

The specific vacuum methods utilized are: (1) extracting excess water from freshly placed concrete prior to set, thereby increasing early strength and enabling early handling of units, (2) holding forms in place by vacuum and (3) handling and placing finished and hardened concrete units by cast-in-place closures formed and quickly hardened by vacuum processes.

The use of these methods results in high strength monolithic construction having exterior and interior finished surfaces, with insulation incorporated into the construction, if desired.

## NEW TYPE OF CONSISTENCY METER TESTED AT ALLATOONA DAM....46-9

**Price 35 cents.**

JAMES M. POLATTY—Oct. 1949, pp. 129-136 (V. 46)

Various mechanical devices have been developed for indicating consistency of concrete since the early days of visual inspection after discharge from the mixer. A new type meter to measure the consistency of concrete while it is being mixed was tested at Allatoona Dam and the operation and results are described.

## EFFECT OF MIXING SEQUENCE ON THE PROPERTIES OF CONCRETE....46-10

**Price 35 cents.**

F. L. FITZPATRICK and W. SERKIN—Oct. 1949, pp. 137-140 (V. 46)

The order in which the constituents of concrete (aggregates, cement and water) are combined in the mixing operation, has a significant effect upon the properties of the concrete as to workability, strength, density, surface finish and absorption. Tests are reported.

## BOND OF CONCRETE REINFORCING BARS....46-11

**Price 35 cents.**

ARTHUR P. CLARK—Nov. 1949, pp. 161-184 (V. 46)

The tests reported were made to compare the resistance to slip in concrete (bond) of deformed bars when tested in beams and companion pull-out specimens, to secure information on the effects of size of bar, the type of deformations on the bars and the strength of concrete on the bond. The bars were cast in a horizontal position in all test specimens. The variables were depth of concrete under the bar, length of embedment of the bar in the concrete, strength of concrete and diameter of bar. Slip of the bar was measured at the loaded and free ends. Three tests were made with 2 in. of concrete under the bar, with 15 in. of concrete under the bar and with 3-in., 12-in., and 16-in. embedments.

Bond strengths for the beams and the pull-out specimens were affected similarly by changes in the geometry of the bars and the bond test specimens. They were greater when the bars were near the bottom than when they were near the top of the specimens. The highest bond strengths were obtained with bars having deformations conforming to suggested requirements for maximum spacing and minimum height and providing ratios of shearing to bearing areas less than 10, usually less than 6.

## PERLITE AGGREGATE: ITS PROPERTIES AND USES....46-12

**Price 35 cents.**

J. JOHN BROUK—Nov. 1949, pp. 185-192 (V. 46)

Synthetic expanded volcanic rock, better known as perlite is a fairly recent addition to the lightweight aggregate field. Its use in concrete is governed by weight, gradation, mixing procedure, cellular structure, strength of cell walls, insulating properties, cost, etc. Air entrainment appears to be necessary to give a workable, nonsegregating mix. Perlite aggregate blended with sand can be used in high strength structural concrete and concrete products.

## THIN WALL CONCRETE SHIP CONSTRUCTION....46-13

**Price 35 cents.**

FRANCIS R. MAC LEAY—Nov. 1949, pp. 193-204 (V. 46)

The development of construction methods for placing thin concrete walls is described. After several attempts using the standard method of pouring between double forms, a new method was developed which permitted the successful construction of ¾-in. and 1½-in. concrete walls. Tests were also conducted to determine the practicability as well as the strength of Gunite as a medium to unite precast units into a monolithic ship. Later these methods were used successfully in the construction of a concrete landing craft for the United States Navy.

## SPECIFICATIONS SHOULD BE REALISTIC....46-14

**Price 35 cents.**

HARRY F. THOMSON—Nov. 1949, pp. 205-220 (V. 46)

Specifications for concrete in moderate-sized and lesser construction frequently contain provisions which are ambiguous, conflicting in application, or do not fully recognize local materials. Most of these questionable features result from (1) inadequate information regarding the characteristics of concrete, or (2) use of ready-written specifications without revision for conditions or changes in standard requirements.

Among the features discussed are: specifying both method and result; "frozen" specifications; habitual use of "1-2-4", multiple provisions for quality; strength without naming consistency; recognition of local materials; use of compression tests; time of placing; bin-test of cement. Numerous quotations from specifications are given, and suggestions offered for bringing the provisions in line with actual conditions.

## INSPECTION AND TESTING OF MATERIALS....46-15

**Price 35 cents.**

NICOLAAS T. F. STADTFELD—Dec. 1949, pp. 237-248 (V. 46)

This paper sets forth the care taken in the inspection and testing of 7 million barrels of portland cement used in 4 million cu yd of concrete of great uniformity and every indication of durability after 12 years of observation. It stresses for the concrete the importance of low water solubility, freedom from "laitance," lack of cement burns of the workers, low alkali content, and it shows above all the necessity for manufacturing control of the cement clinker. It describes how premature set of concrete was prevented. It also deals with the inspection, grading, and testing of aggregates. The Board of Water Supply specifications for aggregates are given as well as other factors used in securing good material.

## FLEXURE OF CELLULAR SHELLS...46-16

**Price 35 cents.**

F. E. WOLOSEWICK—Dec. 1949, pp. 249-256 (V. 46

After a brief discussion of the uses of cellular shells and methods available for the solution of design problems, a sample problem is set up using the theorem of least work. From charts the moments are determined and the effect of added stiffeners is assessed.

## CEMENT PERFORMANCE IN CONCRETE EXPOSED TO SULFATE SOILS....46-17

**Price 35 cents.**

L. A. DAHL—Dec. 1949, pp. 257-272 (V. 46)

The Long-Time Study of Cement Performance in Concrete deals with the performance of portland cements in concrete under various conditions of exposure in the field. Among these conditions is exposure to sulfate soils, that is, to the so-called "alkali soils." This part of the investigation is reported in Chapter 5, in which the complete data are given. The present paper describes briefly the work reported in Chapter 5 and the conclusions which have been drawn. Those readers who wish to study the results in greater detail are referred to the more complete report.

## EARLY STRENGTH OF CONCRETE AS AFFECTED BY STEAM CURING TEMPERATURES....46-18

**Price 35 cents.**

JOSEPH J. SHIDELER and WILBUR H. CHAMBERLIN —Dec. 1949, pp. 273-284 (V. 46)

The testing and results obtained on 990 6 x 12-in. concrete cylinders steam cured for various periods and at temper-

atures ranging from 100 to 200 F are discussed. Strength results are given for ages ranging from 6 hours to 28 days and strengths of companion specimens moist cured at 70 F are given whenever possible. The information presented was obtained for the Bureau of Reclamation for writing specifications covering the steam curing of precast units such as irrigation pipe.

## A SHORTCUT FOR DETERMINING REINFORCEMENT IN REINFORCED CONCRETE.....46-19

**Price 35 cents.**

V. BOGVAD-CHRISTENSEN—Dec. 1949, pp. 285-292 (V. 46)

This paper presents a graph which gives the complete relationship between moments, thrusts, concrete dimensions, reinforcement and resulting stresses for the reinforced concrete members in question.

## PRECAST UNITS FOR SHORT-SPAN BRIDGES.....46-20

**Price 35 cents.**

ROBERT C. HANCKEL—Jan. 1950, pp. 317-328 (V. 46)

Bridge replacement in Lowell, Mass., where minimum traffic interruption was necessary, led to the adoption of precast reinforced concrete units. Available mobile hoisting equipment limited the maximum weight of sections to about 12 tons which permitted small bridges to be precast as complete structures, while for larger bridges, subassemblies were precast and incorporated into the bridge by a cast-in-place deck. The precasting procedures and construction processes are described and illustrated.

## INFLUENCE OF SUBGRADES AND BASES ON DESIGN OF RIGID PAVEMENTS.....46-21

**Price 35 cents.**

KENNETH B. WOODS—Jan. 1950, pp. 329-348 (V. 46)

This paper has been developed from research data, published reports, and experiences gained by observing the performance of both rigid and flexible pavements—particularly as related to subgrade soil textures and the use of base courses. Structural failures of rigid pavements, caused by large concentrations of exceptionally heavy loads, indicate the need for an evaluation of subgrades and bases in determining the most economical design of rigid slabs.

Indications are that the structural capacity of rigid pavements can be improved by the use of location procedures which utilize the best in topographic position and subgrade soil textures. For inferior situations—in regard to position and soils—the use of base courses must be evaluated against the economy of using slabs of increased thickness, more reinforcing steel, or combinations of the two.

It is concluded that it is not entirely feasible, with the present state of knowledge, to standardize rigid pavement design. Rather, the available data indicate that design practices should be developed by regions in which the subgrade soil, availability and type of base course materials, climatic conditions, and traffic volumes and loads are evaluated.

## INSPECTION OF MASS AND RELATED CONCRETE CONSTRUCTION.....46-22

**Price 35 cents.**

LEWIS H. TUTHILL—Jan. 1950, pp. 349-360 (V. 46)

This paper points out that inspection of any kind can be no more effective than that permitted by the specifications and, particularly, by the established job standard of inspection. No distinction is made between the usual activities of inspection and those associated with concrete control since both have the same objective: a serviceable and presentable structure. Details of requirements and procedure to this end are discussed.

## EXPERIMENTAL GROUTING INVESTIGATION FOR CHIEF JOSEPH DAM.....46-23

**Price 35 cents.**

J. M. WELLS—Jan. 1950, pp. 361-376 (V. 46)

This paper describes the development of a method for the control of seepage into the excavation areas of the right abutment, and through the right abutment, at the Chief Joseph Dam project on the upper Columbia River in the State of Washington.

Laboratory studies are presented on various types of grout mixtures, as well as techniques and procedures for drilling in coarse gravel by jetting methods, procedure for grouting pervious gravels and the efficacy of grouting treatment.

## USE OF POZZOLANS IN CONCRETE.....46-24

**Price 35 cents.**

RAYMOND E. DAVIS—Jan. 1950, pp. 377-384 (V. 46)

After stating the characteristics of pozzolonic materials, the effects of replacing with pozzolans part of the portland cement in concrete are considered briefly. Results of tests with fly ash and superfine diatomite are cited to show the possible use of these materials in the East and Midwest where natural pozzolans of the West are not economically available.

## CONSTRUCTION OF LONG-SPAN CONCRETE ARCH HANGAR AT LIMESTONE AIR FORCE BASE.....46-25

**Price 35 cents.**

JOHN E. ALLEN—Feb. 1950, pp. 405-416 (V. 46)

The reinforced concrete arch hangar at Limestone Air Force Base is noteworthy for its size, being one of the largest of this type yet constructed in this country. The superstructure is composed of a thin reinforced arch shell, shaped like an inverted catenary. Construction procedures are described.

## VOLUME CHANGES IN SMALL CONCRETE CYLINDERS DURING FREEZING AND THAWING.....46-26

**Price 35 cents.**

RUDOLPH C. VALORE, Jr.—Feb. 1950, pp. 417-436 (V. 46)

The volume-temperature behavior of small concrete cylinders was observed, using a new mercury-displacement dilatometer, during freezing and thawing cycles in which the range 40 to −20 F was traversed at various rates. Specimens cast from a mix containing 6 bags of portland cement per cu yd, plain and modified by the addition of an air-entraining agent, were tested in air-dry, vacuum-saturated and partially saturated conditions.

Volume-temperature relationships for air-dry specimens yielded uniform thermal expansion data, but those for vacuum-saturated specimens showed departures in the form of transient expansions during freezing, and residual expansions following thawing of the order of 0.8 and 0.4 percent, respectively, a single slow cycle produced decreases in dynamic $E$ exceeding 60 percent.

Much smaller departures were observed for partially saturated specimens (65 to 85 percent of vacuum-saturation) and included, in addition to transient and residual expansions, secondary effects of freezing termed "shrinkage" and relaxation phenomena. The magnitude of the transient and, residual expansions appeared to depend upon the rate of cooling and heating, the degree of saturation of the specimen, and, during the slow cycle, upon the degree of supercooling before freezing.

## INSPECTION AND CONTROL OF CONCRETE FOR HIGHWAY AND BRIDGE CONSTRUCTION.....46-27

**Price 35 cents.**

H. W. RUSSELL—Feb. 1950, pp. 437-444 (V. 46)

Inspection procedures and personnel assignments used by the Illinois Division of Highways for large, medium and small jobs are described. The importance of assurance that materials used are adequately tested and approved before shipment, determination of proper proportions by trial mixtures, control of air entrainment, correction of batch weights and proper preparation and testing of concrete specimens are emphasized. The inspection of ready-mixed concrete supplied to small jobs poses a problem still lacking an adequate solution.

## SOME APPLICATIONS OF ELECTRIC SR-4 GAGES IN REINFORCED CONCRETE RESEARCH.....46-28

**Price 35 cents.**

EIVIND HOGNESTAD and IVAN M. VIEST—Feb. 1950, pp. 445-456 (V. 46)

The writers describe some applications of SR-4 gages in reinforced concrete research at the University of Illinois. Special emphasis is given to the problems involved in waterproofing gages attached to steel which is later embedded in concrete.

## THE ULTIMATE STRENGTH OF REINFORCED CONCRETE BEAMS...46-29

**Price 35 cents.**

S. D. LASH and J. W. BRISON—Feb. 1950, pp. 457-472 (V. 46)

The results of tests to failure on 57 small reinforced concrete beams are presented, and it is shown that they can be explained satisfactorily on the basis of a simple plastic theory of design. Formulas are proposed for determining the moments of resistance and the critical percentages of reinforcement.

## A WAY TO BETTER PAVEMENT CONCRETE.....46-30

**Price 35 cents.**

F. H. JACKSON—Mar. 1950, pp. 489-496 (V. 46)

This paper discusses the performance requirements of concrete pavements from the standpoint, primarily, of the quality of the concrete as a material. It is pointed out that the lack of durability of much of our present day concrete may be due to our methods of construction. The writer believes pavement durability could be significantly improved by using a scientifically proportioned mix of dry consistency, well compacted by vibration or tamping, and with close control over aggregate gradation, in place of the oversanded plastic mixtures which we now use. He does not believe that air entrainment is necessarily the final answer to the problem of surface deterioration but that we should seriously consider overhauling our entire construction practice as well as our present methods of controlling the uniformity of aggregate gradations.
Current practice in the construction of concrete pavements and airport runways in Great Britain and past practice in Germany in the construction of the autobahnen are discussed to show that the placing of harsh, dry mixtures with close control of aggregate gradations is entirely feasible from the construction standpoint.

## DESIGN AND CONSTRUCTION OF A CIRCULATING WATER INTAKE..46-31

**Price 35 cents.**

W. S. COLBY—Mar. 1950, pp. 497-508 (V. 46)

The author describes the design and construction of a 250,000 gpm power plant intake. An interesting feature of the pump well cofferdam was the reinforced concrete ring wales, which resulted in savings in operations because the interior of the cofferdam was kept clear for excavating, driving of drilled-in caissons, tremie operations and placing of the pump well concrete.

## THE CONTRACTOR'S VIEWPOINT OF INSPECTION.....46-32

**Price 35 cents.**

DONALD C. ANDREWS and NOMER GRAY—Mar. 1950, pp. 509-512 (V. 46)

The interests of owner and contractor are compared and recommendations are made on specifications, personal qualifications of the inspector, and methods of minimizing friction with the contractor.

## CORROSION PROTECTION OF THIN PRECAST CONCRETE SECTIONS.....46-33

**Price 35 cents.**

D. H. PLETTA, E. F. MASSIE and H. S. ROBINS—Mar. 1950, pp. 513-528 (V. 46)

A new electrical resistance technique for measuring the rate of corrosion of steel reinforcing is described. The method employs a thin ribbon 0.008 x 0.25 in. as the resistance element embedded in 6 x 12 in. thin panels, and a Kelvin bridge sensitive to 0.0001 ohm. The data plotted in dimensionless parameter form enable the *half-life* of concrete to be determined at a comparatively early age. The term half-life is defined as the time required for the cross-sectional area of the reinforcing to decrease by one-half its original value due to corrosion. Six mixes, three water-cement ratios, four exposure conditions and three depths of cover were examined.

## DESIGN OF HEXAGONAL BINS...46-34

**Price 35 cents.**

PAUL ROGERS—Mar. 1950, pp. 529-540 (V. 46)

The analysis of hexagonal bins is presented with a practical example, showing the details of computations for lateral pressures, inward pull, hanging loads, diaphragm action and the design of the walls, beams and columns.

## PRECAST CONCRETE IN BRITAIN..46-35

**Price 35 cents.**

P. G. BOWIE and A. R. GOLLINS—Mar. 1950, pp. 541-556 (V. 46)

Precast frames for airplane hangars, garages and farm buildings are discussed. Several systems using precast units in housing construction are described and illustrated. Prestressed precast railroad ties and transmission-line poles are mentioned briefly.

## THE ZIG-ZAG COURSE OF CONCRETE PROGRESS.....46-36

**Price 35 cents.**

HERBERT J. GILKEY—Apr. 1950, pp. 573-580 (V. 46)

Retiring ACI President Gilkey emphasizes that concrete has not become more complex, but rather that as knowledge advances, usually in a zig-zag course, not a straight line, more problems and seemingly contradictory half-truths become evident.

## CONCRETING ON THE OTTAWA RIVER PROJECTS OF THE HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO.....46-37

**Price 35 cents.**

A. L. MALCOLM and R. B. YOUNG—Apr. 1950, pp. 581-596 (V. 46)

Construction procedures on three Hydro-Electric Power Commission of Ontario dam projects are described includ-

ing the placing of concrete in lifts up to 50 ft, material and concrete handling systems, classes of concrete, sand blending and aggregate production, treatment of joints, concrete proportioning, winter concreting, and concrete control measures. A discussion of the merits of high lifts in mass concrete structures concludes the paper.

## PROPOSED DESIGN SPECIFICATIONS FOR TWO-WAY FLOOR SLABS.....46-38

**Price 35 cents.**

N. M. NEWMARK and C. P. SIESS—Apr. 1950, pp. 597-608 (V. 46)

A new design specification for two-way concrete floor slabs is presented. It is proposed as a replacement for the methods currently contained in Section 709 of the ACI Building Code (ACI 318-47). This new specification is based on analyses of continuous rectangular slabs carrying a uniformly distributed load. Account is taken of the continuity of the slab, of the torsional stiffness of the beams, and of the deflection of the beams.

## INSPECTION OF BUILDING CONSTRUCTION.....46-39

**Price 35 cents.**

LEONARD E. DUNLAP—Apr. 1950, pp. 609-612 (V. 46)

From the viewpoint of an architect, inspection on a job involving architectural and structural concrete encompasses a number of factors. Accurate plans and specifications, testing and developing the concrete mix design, proper erection of forms and placement of concrete, and systematic curing require close attention by a competent superintendent and qualified inspectors.

## AUSTRALIAN AGGREGATES AND CEMENTS IN RELATION TO CEMENT-AGGREGATE REACTION 46-40

**Price 35 cents.**

A. R. ALDERMAN, A. J. GASKIN, R. H. JONES and H. E. VIVIAN—Apr. 1950, pp. 613-616 (V. 46)

A wide variety of Australian aggregates and cements has been examined with a view to estimating the possible incidence of expansive reaction in concrete made from these materials.

Mortar bars were observed over periods up to two years and the results correlated with petrographic examination of the aggregates and chemical composition of the cements. This correlation has shown that in most cases the potential reactivity can be assessed by petrographic examination but that doubtful material requires supplementary mortar tests.

## SOME AUSTRALIAN STUDIES ON CEMENT-AGGREGATE REACTION IN MORTAR.....46-41

**Price 35 cents.**

H. E. VIVIAN—Apr. 1950, pp. 617-624 (V. 46)

This paper summarizes briefly some of the papers on cement-aggregate reaction which have been published by the Commonwealth Scientific and Industrial Research Organization in Australia. These papers deal with four aspects of mortar expansion; the change in mortar tensile strength as expansion occurs, the effects on mortar expansion of alkali mobility, of void space in the mortar and of different storage conditions.

## CARBON DIOXIDE AND THE CEMENT-AGGREGATE REACTION 46-42

**Price 35 cents.**

A. J. GASKIN—Apr. 1950, pp. 625-628 (V. 46)

Spotting and expansion of cement mortars due to alkali-aggregate reaction can be prevented by treatment of the set mortar with carbon dioxide. Active alkali hydroxides, produced by hydrolysis of cement particles, are thereby converted to carbonates, which are inactive in contact with most aggregates. Sufficient "protective" calcium carbonate can be produced throughout the mortar to confer permanent immunity from spotting and expansion, but no appreciable improvement in tensile strength could be obtained. Carbonated mortars were found to have a high degree of dimensional stability.

## SPACING OF MOMENT BARS IN PRECAST JOISTS.....46-43

**Price 35 cents.**

F. N. MENEFEE and H. L. KINNIER—Apr. 1950, pp. 629-636 (V. 46)

Spacing of moment bars in precast reinforced concrete joists in some cases has been as small as $\frac{1}{16}$ in. although it is probable that most manufacturers use $\frac{3}{4}$-in. shear or diagonal tension bars with corresponding spacing of moment bars.

ACI Building Code places a one in. minimum on spacing. Undoubtedly, the regulations were written with monolithic concrete rather than light precast joists in mind. Spacing has no particular significance if all the requirements for bond are met independently of it.

In a properly designed precast joist, theoretical computations show that bond stresses are always less in proportion to the maximum allowed than are any of the other stresses.

The tests herein reported were an attempt to determine whether the artificial reduction of bond area, up to 30 percent, had any effect on the over-all strength of the joist, and to give some indication as to whether $\frac{3}{4}$-in. spacing of moment bars introduced a controlling weakness. While test results on 14 joists, all of the same dimensions, could hardly produce conclusive evidence, the results, along with the service record of precast joists, indicate the $\frac{3}{4}$-in. spacing of moment bars, with $\frac{3}{4}$-in. maximum size aggregate and with all other requirements of the Building Code met, will give a joist which will fail otherwise than in bond, from which it appears that the present one in. minimum in the current Building Code should be thoroughly studied with a view toward modification for precast joists, with well anchored reinforcement—if opinion and tests justify such a change.

## PROPOSED SPECIFICATIONS FOR MINIMUM BAR SPACING AND PROTECTIVE COVER IN PRECAST CONCRETE FRAMING MEMBERS...46-44

**Price 35 cents.**

ARSHAM AMIRIKIAN—Apr. 1950, pp. 637-640 (V.46)

The use of relatively small size aggregates and favorable conditions for quality control make it possible to place reinforcing bars in precast concrete work at closer spacings than those specified or required in conventionally poured-in-place concrete construction. Better quality control and utilization of rich mixtures make it also possible to obtain corrosion protection of reinforcing with relatively thin covers. Recommendations are given for a new basis of specifying bar spacing and cover, in the form of proposed specifications applicable to precast concrete work, together with a discussion of some of the considerations justifying the suggested specifications.

## THE PROBLEM OF SLAB DIMENSIONS.....46-45

**Price 35 cents.**

SUBCOMMITTEE 2, COMMITTEE 325—Apr. 1950, pp. 641-648 (V. 46)

The dimensions of concrete pavement slabs necessary for optimum results depend on and are influenced by a great many factors, economic as well as physical. The factors and their relationship to slab dimensions discussed include strength, elastic and plastic properties, volume change, subgrades, construction conditions and slab thickness, length and width.

*Continued on p. 58*

# DISCUSSION

## Discussion closed January 1, 1950

Sept. Jl. '49

**Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)**—Report of Committee 616

**An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures**—J. R. Leslie and W. J. Cheesman

**Manufacture of Reinforced Foam Concrete Roof Slabs**—I. T. Koudriashoff

**Suggestions on Concrete Floor Construction**—Ernst Gruenwald

**Use of Air-Entraining Concrete in Canal Lining**—Joseph J. Waddell

## Discussion closed February 1, 1950

Oct. Jl. '49

**The Use of Portland-Pozzolan Cement by the Bureau of Reclamation**—Robert F. Blanks

**Resistance of Concrete and Protective Coatings to Forces of Cavitation**—Walter H. Price and George B. Wallace

**Vacuum Processes Applied to Precast Concrete Houses**—K. P. Billner and Bert M. Thorud

**New Type of Consistency Meter Tested at Allatoona Dam**—James M. Polatty

**Effect of Mixing Sequence on the Properties of Concrete**—F. L. Fitzpatrick and W. Serkin

## Discussion closed March 1, 1950

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**Bond of Concrete Reinforcing Bars**—Arthur P. Clark

**Perlite Aggregate: Its Properties and Uses**—J. John Brouk

**Thin Wall Concrete Ship Construction**—Francis R. Mac Leay

**Specifications Should Be Realistic**—Harry F. Thomson

## Discussion closed April 1, 1950

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**Flexure of Cellular Shells**—F. E. Wolosewick

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**Early Strength of Concrete as Affected by Steam Curing Temperatures**—Joseph J. Shideler and Wilbur H. Chamberlin

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**Inspection of Mass and Related Concrete Construction**—Lewis H. Tuthill

**Experimental Grouting Investigation for Chief Joseph Dam**—J. M. Wells

**Use of Pozzolans in Concrete**—Raymond E. Davis

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**Construction of Long-Span Concrete Arch Hangar at Limestone Air Force Base**—John E. Allen

**Volume Changes in Small Concrete Cylinders During Freezing and Thawing**—Rudolph C. Valore, Jr.

**Inspection and Control of Concrete for Highway and Bridge Construction**—H. W. Russell

**Some Applications of Electric SR-4 Gages in Reinforced Concrete Research**—Eivind Hognestad and Ivan M. Viest

**The Ultimate Strength of Reinforced Concrete Beams**—S. D. Lash and J. W. Brison

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**The Contractor's Viewpoint of Inspection**—Donald C. Andrews and Nomer Gray

**Corrosion Protection of Thin Precast Concrete Sections**—D. H. Pletta, E. F. Massie and H. S. Robins

**Design of Hexagonal Bins**—Paul Rogers

**Precast Concrete in Britain**—P. G. Bowie and A. R. Collins

## Discussion closes August 1, 1950

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**The Zig-Zag Course of Concrete Progress**—Herbert J. Gilkey

**Concreting on the Ottawa River Projects of the Hydro-Electric Power Commission of Ontario**—A. L. Malcolm and R. B. Young

**Proposed Design Specification for Two-Way Floor Slabs**—N. M. Newmark and C. P. Siess

**Inspection of Building Construction**—Leonard E. Dunlap

**Australian Aggregates and Cements in Relation to Cement-Aggregate Reaction**—A. R. Alderman, A. J. Gaskin, R. H. Jones and H. E. Vivian

**Some Australian Studies on Cement-Aggregate Reaction in Mortar**—H. E. Vivian

**Carbon-Dioxide and the Cement-Aggregate Reaction**—A. J. Gaskin

**Spacing of Moment Bars in Precast Joists**—F. N. Menefee and H. L. Kinnier

**Proposed Specifications for Minimum Bar Spacing and Protective Cover in Precast Concrete Framing Members**—Arsham Amirikian

**The Problem of Slab Dimensions**—Subcommittee 2, Committee 325

**Effect of Entrained Air on Bond Between Concrete and Reinforcing Steel**—Eivind Hognestad and C. P. Siess

# "There's no such animal," he cried!

MY FRIEND and I were picking the ponies one day when I started telling him about *a sure thing* I heard about.

"You say it pays four bucks for every three?" he asked. "And can't lose? It *automatically* wins? Must be illegal!"

"Not a bit," I replied. "In fact, the government very much approves..."

"Our government approves of a horse who can't lose..."

"Who said anything about a horse?" I asked.

"So what else could it be but a horse...?"

"It not only could be—but is—U. S. Savings Bonds," was my prompt reply.

"For every three dollars you invest in U. S. Savings Bonds you get four dollars back after only ten years. And if you're a member of the Payroll Savings Plan—which means you buy bonds *automatically* from your paycheck—that can amount to an awful lot of money while you're not looking. Hey, what are you doing?"

"Tearing up my racing form! The horse I'm betting on from now on is U. S. Savings Bonds."

**Automatic saving is <u>sure</u> saving—U.S. Savings Bonds**

# ACI publications in large current demand

## Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

This standard is of large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$3.00, to ACI Members—$1.75.

## ACI Standards—1949

218 pages, 6x9 reprinting current ACI Standards: Building Code Requirements for Reinforced Concrete (ACI 318-47); Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46), six recommended practices: Use of Metal Supports for Reinforcement (ACI 319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); Construction of Concrete Farm Silos (ACI 714-46); Winter Concreting Methods (ACI 604-48); Application of Cement Paint (ACI 616-49); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—ten ACI Standards in one book, $2.50 per copy—to ACI Members, $1.50. The "Detailing Manual" (ACI 315-48) is available separately only at its established price—see above.

## Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-Entraining Portland Cements or Air-Entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-Entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944, $1.25 per copy, 75 cents to ACI Members.

## Air Entrainment in Concrete—Book 2 (1947)

A 204-page compilation of information on air entrainment published after Book 1 (1944), including: "Field Use of Cement Containing Vinsol Resin", and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures," by Charles E. Wuerpel, "Entrained Air in Concrete," a foreword and 14 short papers presented at the 1946 Convention; and discussion of the symposium, reprinted from ACI JOURNALS for September 1945, Feb., June and Dec. Part 2, 1946. $2.25 per copy, $1.50 to ACI Members.

## ACI Manual of Concrete Inspection (1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI Members 75 cents.

## "The Joint Committee Report" (1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. 140 pages, price $1.50—to ACI Members $1.00.

## Reinforced Concrete Design Handbook (1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI Members.

For further information about ACI Membership and Publications (including pamphlets presenting synopses of recent ACI papers and reports) address:

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## Synopses

*Continued from p. 53*

### EFFECT OF ENTRAINED AIR ON BOND BETWEEN CONCRETE AND REINFORCING STEEL.....46-46

**Price 35 cents.**

EIVIND HOGNESTAD and C. P. SIESS—Apr. 1950, pp. 649-668 (V. 46)

This paper reports an experimental investigation of the effect of entrained air on the bond properties of the new Laclede and the Hi-Bond bars, each in ½ in. and ⅞ in. sizes. Bond properties were studied through 108 pull-out tests with a constant bar embedment of 12 diameters. Bars were cast vertically as well as horizontally; in the latter case, with 6 and 30 bar diameters depth of concrete under the bars. A 5000 psi basic concrete mix was used, with air content as a major variable.

Within the scope of these tests it was found that up to the normal air content of 4 to 5 percent, the bond of all bars in all positions was reduced, in terms of percent, less than the flexural and compressive strength of the concrete. Where more than 5 percent air was entrained, however, the bond of the horizontal bars was reduced rapidly, probably due to rising air.

The how and why of the development of improved reinforcement for concrete. Revision of the ACI Building Code is suggested to take advantage of

# New-Style Deformed Reinforcing Bars*

By RAYMOND C. REESE†


## SYNOPSIS

The development of reinforcing bars from the original plain round or square forms to the present improved deformed patterns is traced through the early work of Withey and Abrams down to the 1949 report of Clark's tests. Noting that A.S.T.M. has adopted a standard for deformed bars, the author emphasizes the need for revision of design specifications to take into account the characteristics of the improved bars.


The new-style deformed reinforcing bars with higher, closely-spaced deformations have proved their superiority in bonding to the concrete, in forming a series of narrow, closely-spaced cracks instead of a few wide ones, and in providing the equivalent of "special anchorage" as now set up in the ACI Building Code.

Bars in this country were originally plain round or square, then twisted square bars, and it is only within the last few years that the arrangement of deformations on the bars themselves has been thoroughly studied.

## STUDIES OF BOND

Historically, the first scientific appraisal of bond between steel and concrete was made by M. O. Withey in 1906-07, followed by the tests of Duff A. Abrams‡ reported in 1913, in which he discredited the use of twisted squares because of the small angle the bearing faces made with the axis of the bar. He recommended a bar with the bearing lugs as nearly as possible at 90 degrees to the longitudinal axis and with projections around the circumference about one-tenth of the diameter in height and one-half a diameter apart.

In 1937, George Wernisch§ established that threaded bars had a bond resistance as much as 50 percent higher than the commercial deformed

*Presented at the ACI 46th annual convention, Chicago, Ill., Feb. 21, 1950. Title No. 46-48 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 9, May 1950, *Proceedings* V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1950. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Consulting Engineer, Toledo, Ohio.

‡Abrams, Duff A., "Tests of Bond between Concrete and Steel," *Bulletin* No. 71, University of Illinois Engineering Experiment Station, Dec. 1913.

§Wernisch, George R., "Bond Studies of Different Types of Reinforcing Bars," ACI JOURNAL, Nov.-Dec. 1937, *Proc.* V. 34, p. 145.

bars he used. In 1938, H. J. Gilkey* showed that bond resistance, for concrete above 4000 psi, increases much more slowly than the ultimate strength; that bond is not proportional to the total embedment of the bar but, after some 24 diameters, drops off relatively. Also, in the matter of special anchorage, a bar stressed to full working capacity would, in a length of 15 in., elongate an amount equal to the so-called "initial bond failure." Carl Menzel,† in 1939, investigated especially the effects of various positions of the bar and orientation of the deformations, and was convinced that a bar rigidly held in a horizontal position, with considerable concrete below it, would have a reduced bond because of the shrinkage of the concrete during hardening, which would particularly affect the bent-up ends of truss bars. Menzel's studies resulted in the first new type of bars to be marketed.

World War II directed attention to the conservation of steel and the limitation of its civilian use. In April, 1942, C. A. Willson was instrumental in increasing the working stresses from 20,000 to 24,000 psi. He found designers of ships and barges using stresses as low as 12,000 psi because of the poor bonding qualities. Their problem was not that of ultimate failure but the necessity for limiting sharply the size and spacing of individual cracks, which suggested to Willson that a satisfactory deformed bar should have proper projections not only to limit the end slip but also to tie the concrete together, causing a whole series of closely-spaced, infinitesimal cracks in lieu of a few large, widely-spaced ones. In 1943, some modified types of deformed bar were rolled especially for test; and it was found that some of the new-style deformations were much better than the then-standard commercial bars, and, interestingly enough, considerably better than the threaded bars that had shown up so well previously. This opportunity for improvement caused prompt action.

In November, 1943, the American Iron and Steel Institute established its Committee on Reinforced Concrete Research. Roy R. Zipprodt, its first Research Engineer, during 1944-45, held eighteen regional conferences with professional engineers and architects to discuss needed research projects. At the top of the list were bond and the accompanying diagonal tension. A qualitative study of improved deformations under the direction of Arthur P. Clark,‡ in 1946, used a different method of rating, an "average" value obtained by measuring stress and slip under a variety of loads, totalling the partial slips, and dividing by the number of readings. "Improved" bars proved much more effective than "com-

*Gilkey, H. J., Chamberlin, S. J. and Beal, R. W., "The Bond between Concrete and Steel," ACI JOURNAL, Sept. 1938, *Proc.* V. 35, p. 1.

†Menzel, C. A., "Some Factors Influencing Results of Pull-Out Bond Tests," ACI JOURNAL, June 1939, *Proc.* V. 35, p. 517.

‡Clark, Arthur P., "Comparative Bond Efficiency of Deformed Concrete Reinforcing Bars," ACI JOURNAL, Dec. 1946, *Proc.* V. 43, p. 381.

mercial" ones; bars cast horizontally in the top of a specimen were considerably less effective than those cast near the bottom; and the size and spacing of the projections seemed to be the main elements of high bond resistance. It was now possible to formulate rather accurately the desirable characteristics of improved deformed bars.

## NEW DEFORMED BARS

Prior to 1947, there was no definition of a deformed bar, until, guided considerably by Douglas E. Parsons, a specification, A.S.T.M. A 305-47T, was tentatively accepted and, in June, 1949, was adopted as a full standard, A 305-49, by the American Society for Testing Materials. This specification accurately describes the size, position and spacing of the projections on a bar. Fig. 1 illustrates the five types of "improved" deformed bars now in commercial production. As shown in Fig. 2 deformations must be spaced uniformly along the bar at distances not exceeding seven-tenths of the nominal size of the bar, with the deformations on opposite sides similar in size and shape and placed so that the included angle with respect to the longitudinal axis is not less than 45 degrees. Where the included angle is between 45 and 70 degrees, the deformations shall alternately reverse in direction on each side, or those on one side shall be reversed from those on the opposite side. The length of deformation shall be such that the space between extreme ends of deformations on opposite sides of the bar shall not ex-

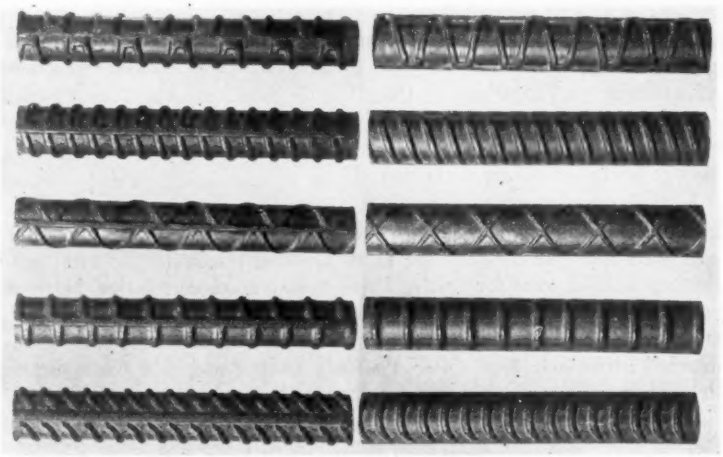

Fig. 1—Types of "improved" deformed bars

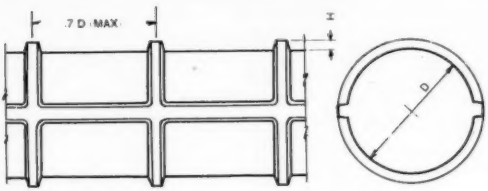


Fig 2—Spacing requirements for deformations

| Nominal size of bar, in. | ⅜ | ½ | ⅝ | ¾ and larger |
|---|---|---|---|---|
| Minimum height H of deformations, percent of nominal size of bar | 4 | 4 | 4½ | 5 |

ceed 12½ percent of the nominal perimeter and the average height of deformation varies slightly, being at least 4 percent for bars ½ in. and smaller, 4½ percent for ⅝ in. and 5 percent for ¾ in. and larger, weights and nominal cross-sectional areas remaining as in the standard table of weights and areas, while rounds of the same weight may replace the square bars. Fig. 3 and Table 1 indicate these same requirements.

Other investigations, such as those of McHenry and Walker* and the still-unpublished tests at Cornell, strove to measure the growth of stress along a bar with SR-4 strain gages, but much work remains to be done in this field. Concurrently, T. D. Mylrea† and others were attempting to rationalize the internal mechanics of a reinforced concrete beam.

Meanwhile, F. E. Richart, as a part of the American Iron and Steel Institute's program of research, repeated and extended Professor Talbot's tests on wall and column footings. These tests‡ clearly indicated that properly deformed bars with hooked ends were no better than straight bars, that the bond values of plain round bars were decidedly less than might have been anticipated, that "commercial" deformed bars varied considerably, that "improved" deformed bars had computed bond stresses at maximum load of from 0.19 to 0.23 $f'_c$, with failures reported mainly as due to diagonal tension.

In November, 1949, Arthur P. Clark§ summarized his tests on quantitative bond values. Test beams were cast both in the normal position, with steel near the bottom, and in an inverted position, with steel near the top turned over for testing. He found good correlation between beam and pull-out tests; that plain round bars do not have any better, and possibly not as good, a value as when tested years ago; that "commercial" deformed bars differed widely from being not much better than plain rounds in some cases to twice that value in others, so that

*McHenry, Douglas and Walker, W. T., "Laboratory Measurements of Stress Distribution in Reinforcing Steel," ACI JOURNAL, June 1948, *Proc.* V. 44, p. 1041.
†Mylrea, T. D., "Bond and Anchorage," ACI JOURNAL, Mar. 1948, *Proc.* V. 44, p. 521.
‡Richart, F. E., "Reinforced Concrete Wall and Column Footings," ACI JOURNAL, Oct. and Nov. 1948, *Proc.* V. 45, pp. 97 and 237.
§Clark, Arthur P., "Bond of Concrete Reinforcing Bars," ACI JOURNAL, Nov. 1949, *Proc.* V. 46, p. 161.

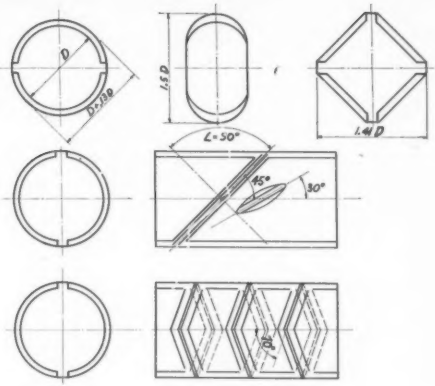


**Fig. 3—Minimum requirements for deformed reinforcing bars**

"deformed" had little significance; and that "improved" bars meeting A.S.T.M. Standard A 305-49 greatly exceeded all other bars.

## REVISIONS IN DESIGN PRACTICE

The U. S. Department of Commerce Simplified Practice Standard R26 is being revised and will presumably be about in the form of Table 2 with bars numbered in eighths of an inch (numbers based upon weight per foot rather than cross-sectional area) and with round bars of equal weight substituted for old type square bars.

**TABLE 1—DIMENSIONAL REQUIREMENTS FOR DEFORMED STEEL BARS FOR CONCRETE REINFORCEMENT—A.S.T.M. DESIGNATION A 305-49§**

| | | Nominal dimensions round sections | | | Requirements of deformations | | |
|---|---|---|---|---|---|---|---|
| Bar No. † | Unit wt. lb/ft | Diameter, in. | Cross sectional area, sq in. | Perimeter | Max. ave. spacing, in. | Min. height, in. | Max. gap, in.‡ |
| 3 | 0.376 | 0.375 | 0.11 | 1.178 | 0.262 | 0.015 | 0.143 |
| 4 | 0.668 | 0.500 | 0.20 | 1.571 | 0.350 | 0.020 | 0.191 |
| 5 | 1.043 | 0.625 | 0.31 | 1.963 | 0.437 | 0.028 | 0.239 |
| 6 | 1.502 | 0.750 | 0.44 | 2.356 | 0.525 | 0.038 | 0.286 |
| 7 | 2.044 | 0.875 | 0.60 | 2.749 | 0.612 | 0.044 | 0.334 |
| 8 | 2.670 | 1.000 | 0.79 | 3.142 | 0.700 | 0.050 | 0.383 |
| 9* | 3.400 | 1.128 | 1.00 | 3.544 | 0.790 | 0.056 | 0.431 |
| 10* | 4.303 | 1.270 | 1.27 | 3.990 | 0.889 | 0.064 | 0.487 |
| 11* | 5.313 | 1.410 | 1.56 | 4.430 | 0.987 | 0.071 | 0.540 |

*These sections have the same weight and area as bars formerly known as 1" sq., 1⅛" sq. and 1¼" sq.
†Bar numbers are based on number of ⅛ inches included in the nominal diameter of the bar section.
‡Chord of 12½ percent of nominal perimeter.
§Reproduced with permission of the American Society for Testing Materials.

**TABLE 2—PROPOSED REVISION OF SIMPLIFIED RECOMMENDATION R 26-49**

Bar numbers and weights*

| Bar No. | Wt. per ft, lb | Bar No. | Wt. per ft, lb |
|---|---|---|---|
| †2 | 0.167 | 7 | 2.044 |
| 3 | 0.376 | 8 | 2.670 |
| 4 | 0.668 | ‡9 | 3.400 |
| 5 | 1.043 | ‡10 | 4.303 |
| 6 | 1.502 | ‡11 | 5.313 |

Table of Equivalent Round Plain (not Deformed) Bars§

| Bar No. | Wt. per foot, lb | Nominal round diameter, in. | Nominal perimeter, in. | Nominal cross-sectional area, sq in. |
|---|---|---|---|---|
| †2 | 0.167 | 0.250 | 0.785 | 0.05 |
| 3 | 0.376 | 0.375 | 1.178 | 0.11 |
| 4 | 0.668 | 0.500 | 1.571 | 0.20 |
| 5 | 1.043 | 0.625 | 1.963 | 0.31 |
| 6 | 1.502 | 0.750 | 2.356 | 0.44 |
| 7 | 2.044 | 0.875 | 2.749 | 0.60 |
| 8 | 2.670 | 1.000 | 3.142 | 0.79 |
| ‡9 | 3.400 | 1.128 | 3.544 | 1.00 |
| ‡10 | 4.303 | 1.270 | 3.990 | 1.27 |
| ‡11 | 5.313 | 1.410 | 4.430 | 1.56 |

*Bar numbers are based on the number of ⅛ inches included in the nominal diameter of the section.
†Bar number 2 in plain rounds only.
‡Bars numbered 9-10-11 correspond to former 1" sq., 1¼" sq., and 1¼" sq. sizes, and are equivalent to those former standard bar sizes in weights and nominal cross-sectional areas.
§This table is for information purposes only and is not part of the Simplified Practice Recommendations.

Before summarizing the advantages of these new-style deformations, a few figures may illustrate some of the items that have been learned. Fig. 4 shows the difference in bond between round bars and square ones, the former having radial stresses, the latter special conditions at the corners. Fig. 5 pictures the recorded strains as measured along the length of hooked bars, the center figure indicating that with new-style deformed bars hooks are practically useless but that with plain round bars (left) considerable stress was picked up by the hook. Fig.

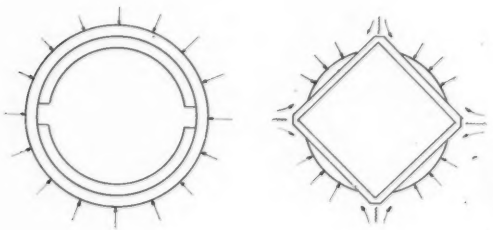

**Fig. 4—Anchorage comparison between round and square bars**

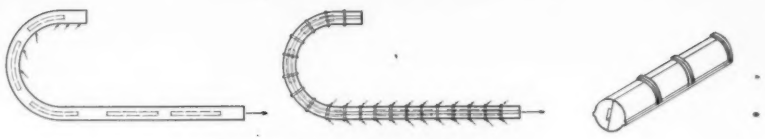

Fig. 5—Recorded strain in hooked bars

6 illustrates the need for closely-spaced deformations, the lefthand view for lugs too far apart to develop the bar, while on the right closely-spaced deformations pick up increments of the load.

Fig. 7 shows in the upper diagram the need for a 40-diameter embedment with wide spacing of lugs. The middle view shows why a 20-diameter embedment with closely-spaced lugs serves equally well. The bottom diagram shows only one side of the problem, the formation of fairly large, widely-spaced cracks that occur with old-style deformed bars, even if specially anchored. Hairline cracks spaced fairly closely together result from use of the new-style deformations, even without hooking of the bar.

The advantages of these new bars, substantiated by the numerous testing programs, include:

1. Greatly improved bond resistance, permitting safe working stresses of some 10 percent of the ultimate concrete strength.

2. The equivalent of "special anchorage" within the meaning of the ACI Building Code, permitting higher values in diagonal tension.

3. An accurate crack control, since the higher projections and closer spacing of deformations prevent the formation of a few large cracks and make the concrete and steel work more closely as a unit.

4. A possible increase in tensile stresses due to more positive bonding and a continuous bonding all along the bar instead of a special anchorage

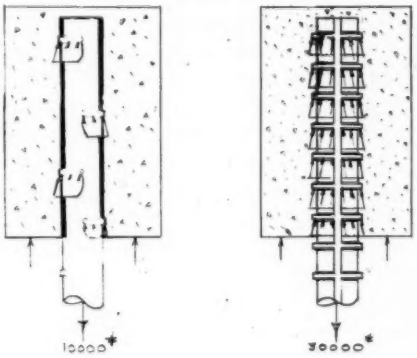

Fig. 6—Effect of spacing of deformations on developed load in reinforcing bars

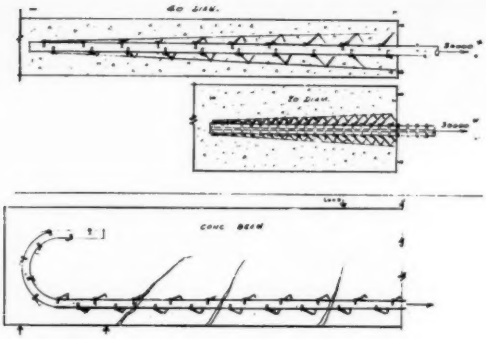

Fig. 7—Effect of spacing of deformations on required embedment of reinforcing bars

at the end. The British Standard Code of Practice "CP114" (1948) allows half the guaranteed yield point but not more than 27,000 psi on their steel reinforcement.

## CONCLUSION

It now appears that our own ACI Building Code Committee 318 must formulate the results of these improvements into design specifications reflecting the greater values to be obtained with the improved bars. It is likely that these will follow the recommendations of ACI Committee 208* for revisions in permissible bond stresses. Account will be taken for the first time of the position of the bar in the concrete during pouring and we may well want to decrease values for plain bars and those not up to the A 305-49 requirements, while materially increasing the values for improved bars.

One outgrowth from the improved bar will be better structures through the closer spacing of much narrower cracks in tension concrete. This is of advantage not only in concrete barges, tanks and similar vessels but also in rendering much less unsightly the conditions of highly stressed negative moment zones around supports.

Beyond this, still further progress is envisioned, possibly coming more slowly; but with improved bars affording better resistance to diagonal tension, some change is likely in that field; and with both bond and diagonal tension well taken care of, it may be possible to raise working stresses in tension and also to use ultimate design methods.

*See "Report of ACI Committee 208 on Bond Stress," p. 677 and ACI *News Letter*, Apr. 1950, p. 20.

Discussion of a paper by Raymond C. Reese:

# New-Style Deformed Reinforcing Bars*

By ERLING REINIUS and AUTHOR

By ERLING REINIUS†

The history of the development of reinforcing bars in the United States from plain round bars to the new-style deformed reinforcing bars has been treated in an interesting manner. In his conclusions the author asks for new design specifications reflecting the greater values to be obtained with the improved bars.

The dimensional requirements for the new deformed reinforcing bars agree closely with the Swedish requirements for deformed reinforcing bars. It is interesting to note that the research work in Sweden and in the United States has yielded approximately the same results and the same conclusions. The new deformed bar has been used in Sweden for eight years. A short article by the writer on these bars appeared in *The Engineer*, Dec. 6, 1946.

It has long been well-known that the bond between concrete and steel is firmer when deformed bars are used than with plain bars. Experience in Europe has shown, however, that the bond of plain bars is sufficient and gives good results provided that steel with low yield points and low allowable stresses are used. Plain bars are also cheaper than deformed bars.

Before the war a great part of the reinforcing steel used in Sweden was imported from Germany. In the course of the war, this import stopped and, consequently, there was a considerable shortage of reinforcing steel. To overcome this difficulty it was suggested that steel with a higher yield point and higher allowable stresses should be used, thereby saving steel.

Obviously, the use of high steel stresses will involve higher bond stresses. Therefore, to attain an efficient utilization of high steel stresses the bond resistance of the bar had to be increased. If comparatively wide cracks are allowed, plain bars with high steel stresses may be used, but in such a case the end anchorage must be satisfactory.

*ACI JOURNAL, May 1950, *Proc.* V. 46, p. 681. Disc. 46-48 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

†Professor of Hydraulic Engineering, Chalmers Institute of Technology, Gothenburg, Sweden, and Consulting Engineer of Vattenbyggnadsbyran (VBB), Stockholm.

In collaboration with A. Frey Samsioe, the writer designed a new deformed reinforcing bar suitable for high working stresses and with so high a bond resistance that end hooks were unnecessary. After a conference with a steel factory (Smedjebackens Valsverks A.B.) it was decided to use a coal steel with a yield point of 57,000 psi. The new bar was tested by the Concrete Commission of the Civil Service Departments and approved in October, 1942.* The specifications of the new bar were, and still are, as follows:

Maximum spacings between the deformations are to be 0.8 of the diameter of the bar. Over a maximum length of four times the diameter of the bar, the sums of the projected areas of the deformations on a plane perpendicular to the longitudinal axis is to be equal to the cross-sectional area of the bar. The lateral sides and the directions of deformations are to include an angle with the longitudinal axis of the bar of not less than 45 degrees.

The new American bar according to ASTM Standard A 305-49 does not fully satisfy the Swedish specification since the sum of the projected areas of deformations is equal to the cross-sectional area over a length of 6.3 times the diameter of the bar for ½-in. bars and 5 times the diameter of the bar for 1-in. bars. The Swedish bar has higher deformations than the new American bar and since the size and spacing of the deformations seem to be the main elements of high bond resistance it is probable that the Swedish bar has higher bond resistance. Nevertheless, the design of the two types of bars agrees closely since the Swedish bar is used with higher working stresses and this necessitates a higher bond resistance than required for the American bar.

The Swedish Concrete Commission has been working on a new code of practice since 1940 and has made extensive investigations of concrete reinforced with plain and with deformed bars of high-quality steel (yield point up to 86,000 psi). A great number of beams and pull-out specimens have been tested with static as well as with pulsating loads. Some of the results are published in Swedish papers. The new code† contains, among other things, the following stipulations:

*Steel quality*—yield point: 57,000 psi for bars with diameter less than 18 mm (0.7 in.); 54,500 psi for bars with diameters between 18 and 30 mm (1.2 in.); and 52,000 psi for bars with diameter more than 30 mm. Allowable steel stress at static load: 31,500 psi for bars with diameter less than 16 mm and 28,500 psi for bars with diameter between 16 and 32 mm. At dynamic loads (*i.e.*, bridges) the allowable stresses are 28,500 and 27,500 psi, respectively. These stresses are allowed only if the concrete has a cube strength at 28 days of more than 3500 psi. If the cube strength is between 2850 and 3500 psi, the allowable steel stresses must be lowered by 2800 psi. Allowable bond stress varies from 160 to 240 psi according to the variation of the cube strength of the concrete from 2850 to 5700 psi. These cube strengths correspond to cylinder strengths of about 2000 to 4000 psi. (Thus the allowable bond stresses in Sweden agree closely to the bond stresses recommended for top bars by ACI Committee 208.) If end anchorage is satisfactory, bond stresses may be raised by 50 percent. End hooks are not necessary. Overlapping of bars: 30-35 diameters of the bar. Thickness of covering concrete: 1.5 diameters in height and 2 diameters in lateral direction; lateral distance between bars: 2 diameters and distance between different layers of bars: 1.5 diameters. Bending radius of bars: 10 diameters.

**Tillagg nr 1 till Statliga Cement—och Betongbestammelser av Ar 1934* (Supplement No. 1 to the State Code of Practice of Cement and Concrete of 1934), Stockholm, 1942.

†*Statliga Betongbestammelser, Del 1*, (State Code of Practice of Concrete, Part 1), Stockholm, 1949.

During the years 1942 to 1950 the use of the new deformed bars has risen year by year. Last year about 40 percent of all reinforcing steel used in Sweden consisted of these bars. Nearly half of all reinforcing steel used in Sweden is imported; thus the reinforcing steel manufactured in Sweden is mainly the new type. In Norway, Finland and other countries, the Swedish type of deformed bars is manufactured and the reason for this rapidly increasing use is the economic advantage. In comparison with plain round bars with a yield point of 37,000 psi and allowable stresses of about $\frac{2}{3}$ the allowable stresses for deformed bars, the saving in steel quantity and in cost is conderable.

One of the steel factories (Smedjebackens Valsverk A.B.) is also rolling reinforcing bars with the same type of deformations but of high-quality steel with a yield point of 86,000 psi and with diameters less than 14 mm (0.55 in.). These bars have only been used as reinforcement in slabs. Tests have shown that a working stress of 38,000 psi gives sufficient margin of safety against anchorage failure if the diameter is less than 14 mm. However, this quality has not been approved by the Concrete Commission for normal use. Some new deformed bars with a yield point of 57,000 psi have been used successfully. However, some precautions have to be taken.

When cutting this hard steel, the ordinary edges of the cutting machine were too weak and consequently wear and tear were high. When using edges of a higher quality, the cutting has been effected without trouble.

On several occasions it was observed that the bars sometimes broke when bent. An investigation showed that in these cases the bending had been done to a radius of about 2 diameters which is considerably sharper than the radius of 10 diameters which is the stipulated minimum in the code of practice. In some cases the bars had been bent both forwards and backwards. When the rules pertaining to the radius were followed no breakage was observed. An investigation has shown that even bars bent to a small radius have maintained normal values of yield point and breaking point.

For work on the site, deformed bars have shown many advantages over plain bars, the most important being that end hooks are not necessary and that the bars, when placed, remain in position and do not slip. The deformed bars do not get crooked in handling as easily as bars with a low yield point. Consequently, it has been possible to use deformed bars with more slender dimensions than those suitable for plain bars.

## AUTHOR'S CLOSURE

The current design practices in Sweden have been well summarized. It is interesting that their researches produced a bar with deformations so closely parallel to those developed independently in this country. This would seem to indicate that the deformations called for in ASTM A 305 are well designed to develop maximum bond and to spread the cracks in the concrete into a whole series of narrow openings.

Reinius also mentions that high yield-point steels are available in Sweden and that fairly high working stresses are permitted, especially under static load conditions. It has been the experience in the United States that no particular benefit or economy resulted in the use of high working stresses with high yield-point steels unless they were coupled with high-grade concrete having compressive cylinder strengths of 3750 psi and up.

Are minor cracks cause for rejection of precast concrete members?

# Extent and Acceptability of Cracking in Precast Concrete Framing Members*

By ARSHAM AMIRIKIAN†


## SYNOPSIS

One of the important questions arising in precast reinforced concrete construction concerns the acceptability of cracked framing members. Owing to the absence of definite guides for inspection, the fabricator often is penalized by unreasonable rejections. As an aid in this matter, a proposal is made for an acceptability clause which gives practical definitions of cracks and specifies limitations for acceptability.


## INTRODUCTION

By its nature, concrete is susceptible to cracking. Regardless of the source or cause, when the tensile strain in concrete becomes greater than that which the concrete will withstand, failure or cracking will result. The stress at which failure occurs is determined by flexural tests and is called the modulus of rupture. In practice, this value is generally taken as 15 to 20 percent of the compressive strength of concrete. Owing to this relatively small tension resistance, cracks of varying extent are to be expected in most types of concrete structures.

In conventional or poured-in-place concrete construction, cracking does not constitute too serious a problem. For one thing, no significant cracks develop during construction to cause concern in inspection. Cracks which develop later under service conditions are generally taken as natural occurrences, something about which little can be done. This attitude usually is justified by another consideration in design, by which no reliance is placed on the tension value of concrete, since the strength of concrete framing elements are computed on the basis of cracked sections.

Cracking in precast work will generally occur before final placement of the member. It may develop during casting, curing, handling and

*Presented at the ACI 46th annual convention, Chicago, Ill., February 21, 1950. Title No. 46-49 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 9, May 1950, *Proceedings* V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1950. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Head Designing Engineer, Bureau of Yards and Docks, Navy Dept., Washington, D. C.

erection operations; thus, it is primarily a construction problem which requires criteria of acceptability in inspection.

## CAUSES OF CRACKING

There are a number of sources of strain which may cause cracking. Some of them are inherent in the design and the adopted fabrication procedure. Others are traceable to poor workmanship and to lack of proper care in the various operations. Still others are due to natural causes, chief of which is shrinkage. Experience gained in the construction of precast concrete structures indicates that, if certain precautionary measures are taken, the extent of cracking may be greatly reduced. These include the following.

### Molds

The use of rigid molds, built of concrete or steel, has one disadvantage in that the rigidity prevents any appreciable deformations of the castings due to shrinkage, and thus introduces large strains comparable to locked-up stresses in a welded framing. For this reason, castings should not be left in their molds longer than necessary to obtain the initial strength required for handling. This should be an important consideration in planning for week-end stoppage of work.

With concrete molds, further precaution is necessary to assure that the interior or contact surfaces are smooth and free of craters caused by the dislodgment of particles from the mold lining. The presence of such depressions may cause the keying of the casting to the mold, resulting in large strains which may cause cracking of the element during removal from the mold.

### Reinforcing

To obtain the most effective aid from the reinforcing in resisting shrinkage stresses, all castings should be reinforced with welded wire fabric of rather close mesh, say not over 2-in. spacing. The use of wire fabric, in lieu of ordinary reinforcing, is particularly important in thin sections, such as thin-shell ribbed panels and cellular framing.

Where the wire fabric and ordinary reinforcing are preassembled in the form of cages, a close dimensional tolerance should be maintained to assure their proper fitting and positioning in the molds. In many instances, when poorly fitted assemblies are forced into the molds, their tendencies to spring up during casting and finishing operations may cause cracking even during initial setting of the concrete.

### Shape and outline of castings

In designing precast members, the use of unsymmetrical or unbalanced sections and of irregular outlines in the form of T's and L's should be

avoided whenever practicable. Such elements are difficult to handle and are subject to excessive shrinkage strains, resulting in cracking.

### Handling of castings

Improper handling of cast elements constitutes a dangerous source of strain. This is particularly serious during the initial handling operations, namely, when removing the castings from the molds. In modern precasting practice where the use of a limited number of molds necessitates a fast turn out in production, the castings have developed very little strength at the time of their removal. In such cases vacuum lifting devices are useful.

Precaution is also necessary during later handling operations involving storage, transfer and erection. The locations of slings or points of support should be so selected as to minimize handling strains. Furthermore, to obviate cracking during these operations, the computed maximum stresses in the castings, considered as homogeneous sections, should be kept well within the safe limits of the tensile strength of the concrete. The developed tensile strength at the various ages of the concrete, corresponding to the time of each handling operation, should be determined by laboratory tests on plain beam specimens.

### Curing

The protection of castings during the curing periods of storage is important. When the castings are stored in the open for air curing, a protective cover such as wet burlap blankets will shield castings against direct sun rays and will greatly reduce the hazards of cracking during that stage.

## ACCEPTABILITY AND REJECTION

The precautionary measures described will unquestionably strengthen the castings and minimize cracking. They will not, however, entirely eliminate all types of cracking. Regardless of changes in procedure or maintenance of all possible care, there will always be some cracking during the many operations between molding and final placement. The practical question is then to define a crack which is unavoidable and harmless, and one which may prove to be harmful or whose presence may be deemed as objectionable.

### Hair cracks

All hair cracks and surface cracks, that is to say, cracks which penetrate only partially through the depth of a section, fall within the first category. These cracks generally develop in the early phases of shrinkage of the concrete and are present in practically all types of concrete structures. Except for appearance, in certain locations, such cracks are of little importance and should not constitute a cause for rejection.

Furthermore, it has been observed in concrete floating structures that cracks of this type often heal or seal off when exposed to water. The sealing apparently takes place when particles of free lime present in the crack hydrate with the passage of water. Most cracks thus sealed remain impervious in subsequent exposure to water.

#### Through cracks

Under additional strain, a hair crack of partial penetration may extend through the whole depth of a casting and thus become a through crack. If the aperture is narrow, say not over 0.01 in., the structural adequacy of the casting will remain unimpaired as long as corrosion of the reinforcement is prevented. For this purpose, the reinforcement may be treated by cement slurry, or by other protective coating before it is put in the molds or forms. Accordingly, castings containing through cracks up to 0.01 in. wide may be accepted, provided the reinforcement is given a protective coating.

#### Fractures

Cracks of the second category may be defined as total cleavages, or fractures, of measurable width through which water could seep more or less freely. Such cracks generally develop from over-strain due to improper handling. While some of these cracks may be repaired and effectively sealed, their acceptability will be governed by the importance and the function of the particular framing under consideration. On the other hand, if these cracks cannot be effectively sealed, because of the prevalence of large strains at the same zones due to service loading, the rejection of the affected castings then becomes an obvious necessity.

## PROPOSED SPECIFICATION CLAUSE

Based on the above discussion, the following is proposed:

Unless otherwise stated in job specifications, the following shall govern the acceptability of castings which have developed cracks during construction:

(a) Where appearance is not an important consideration, precast concrete framing members containing hair cracks may be accepted. (A hair crack is defined as a surface crack of minute width, visible but not measurable by ordinary means.)

(b) Where appearance is not an important consideration, precast concrete framing members containing cleavage cracks may be accepted, provided the reinforcement is given a protective coating before it is placed in the molds. (A cleavage crack is defined as a through crack not over 0.01 in. wide, but which in the judgment of the inspector penetrates at least to the reinforcing steel.)

Further data from a University of Wisconsin research project started in 1936

# Effect of Sustained Loading on Compressive Strength and Modulus of Elasticity of Concrete*

By GEORGE W. WASHA† and PAUL G. FLUCK‡


## SYNOPSIS

This paper presents results which show the plastic flow characteristics of hand-rodded and vibrated concrete over a 10½-year period. Three different cements and three different water-cement ratios were included in the test program. The effect of the 10½-year loading period on the compressive strength and the modulus of elasticity is also shown.


## INTRODUCTION

In 1936 a research project was started in the materials testing laboratory at the University of Wisconsin to obtain a comparison of the properties of vibrated and hand-placed concrete made with different cements, and to determine what results might be expected with mixes much leaner than those commonly used for construction. A report of the results obtained up to 1940 was presented in the ACI Journal.§

The present report gives the long term plastic flow results as well as the results of compression and modulus of elasticity tests made at the end of the 10½-year loading period. The following sections on materials, manufacture, and testing have been largely abstracted from the 1940 report.

## MATERIALS

### Aggregate

The Janesville sand used had a fineness modulus of 2.67, and contained about 60 percent quartz and 30 percent dolomite. The Janesville gravel used had a maximum size of 1½ in., a fineness modulus of 7.10, and consisted largely of dolomitic particles.

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§Washa, George W., "Comparison of the Physical and Mechanical Properties of Hand Rodded and Vibrated Concrete Made With Different Cements," ACI JOURNAL, *Proc.* V. 36, pp. 617-645.

### Cement

The three cements used were designated as follows: 1, a standard portland cement; 2, a high early strength portland cement; 4, a high-silica cement made by grinding together 70 percent portland cement clinker and 30 percent high-silica compound of pozzolanic origin. Tables 1, 2 and 3 give the chemical analyses furnished by the manufacturers, the calculated compound compositions, and the physical properties of the cements as determined at the University of Wisconsin.

**TABLE 1—CHEMICAL ANALYSIS OF CEMENT—MANUFACTURERS' DATA**

| Cement | $SiO_2$ | $Fe_2O_3$ | $Al_2O_3$ | $CaO$ | $MgO$ | $SO_3$ | Loss on ignition |
|---|---|---|---|---|---|---|---|
| 1 | 20.32 | 2.78 | 5.66 | 63.68 | 4.17 | 1.73 | 0.88 |
| 2 | 19.64 | 3.02 | 6.48 | 66.02 | 1.07 | 2.25 | 1.21 |
| 4 | 33.80 | 2.27 | 6.83 | 52.75 | 2.02 | 1.19 | 2.00 |

**TABLE 2—CALCULATED COMPOUNDS**

| Cement | $C_3S$ | $C_2S$ | $C_3A$ | $C_4AF$ | $CaSO_4$ |
|---|---|---|---|---|---|
| 1 | 57.8 | 14.7 | 10.3 | 8.5 | 2.9 |
| 2 | 65.0 | 7.4 | 12.1 | 9.2 | 3.8 |
| 4* | 53.1 | 24.6 | 12.4 | 7.1 | — |

*Compounds present in the clinker. The cement consisted of 70 percent portland cement clinker ground with 30 percent high-silica compound.

## MAKING SPECIMENS

### Vibrator

An internal vibrator with a spud 1¾ in. in diameter, 20 in. long, weighing 7.7 lb was used. The measured speed of vibration of the spud in the concrete was about 3300 rpm.

### Proportions

The mix proportions were determined by the water-cement ratio trial method of proportioning concrete mixtures and are shown in Tables 4 and 5. The vibrated concrete was designed for 0 slump, and the hand-rodded concrete for a 2-in. slump.

### Mixing of concrete

Air dry aggregates were used in all cases, and allowance was made for the water absorbed by the aggregates. All mixing was done thoroughly by hand in metal mixing trays.

### Placing and curing of concrete

The 2-in. slump concrete was hand placed in the molds by rodding in the usual manner. The 0 slump concrete was placed in molds and

**TABLE 3—PHYSICAL TESTS OF CEMENT**

| Property | Cement | | |
|---|---|---|---|
| | 1 | 2 | 4 |
| Normal consistency | 23.5 | 25.0 | 31.0 |
| Initial set | 3 h 30 m | 2 h 30 m | 2 h 0 m |
| Final set | 5 h 30 m | 4 h 0 m | 4 h 30 m |
| Soundness pat | OK | OK | OK |
| Fineness No. 200 sieve | 91.8 | 99.0 | 99.0 |
| Tensile strength of 1:3 standard mortar briquettes, psi | | | |
| 1-day | 160 | 330 | 195 |
| 7-day | 320 | 460 | 405 |
| 28-day | 400 | 450 | 490 |
| 360-day | 370 | 470 | 525 |
| 720-day | 355 | 430 | 545 |
| Compressive strength of 1:2.75 mortar, 2-in. cubes, psi | | | |
| 7-day | 3145 | 5000 | 4125 |
| 28-day | 4820 | 6155 | 6700 |
| 360-day | 5745 | 6615 | 7365 |
| 720-day | 5160 | 6210 | 7400 |

was vibrated approximately one minute. The concrete was moist cured for 28 days.

## TESTING

Six 6 x 12-in. cylinders were made for each variable condition. Two were tested for ultimate compressive strength at 28 days. Two were loaded to one third of the ultimate strength of the broken cylinders and the modulus of elasticity values determined with a dial compressometer. These cylinders were then used as control (shrinkage) cylinders in the plastic flow tests. The last two cylinders were placed in plastic flow assemblies of the type shown in Fig. 1 and were loaded to one fourth of the ultimate strength of the broken cylinders or to the maximum spring force in the case of the stronger concrete. The spring of the assembly was initially compressed in a testing machine to the proper load. The tie rod nuts were then turned down to hold the spring in the compressed condition so that the assembly with the loaded cylinder in it could be removed from the machine and stored. The cylinders were fitted with reference plugs to allow strain readings with a Berry strain gage at intervals during the loading period. The loads on the cylinders remained practically constant for the entire loading period of 10½ years. The specimens in the plastic flow assemblies and the control specimens were stored in a room in which the

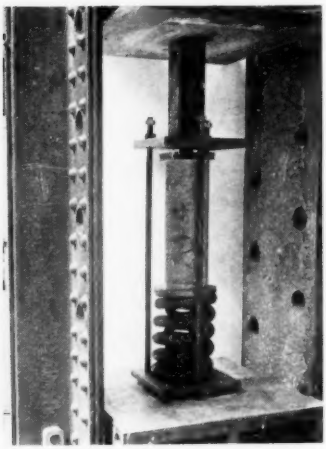

**Fig. 1—Concrete cylinder in plastic flow assembly being subjected to initial loading**

temperature varied from 60 to 80 F, and the relative humidity varied from 25 to 60 percent.

After 10½ years of sustained loading, the cylinders were removed from the load frames and the strain or recovery due to unloading was measured. Measurements of strain were continued for six months after unloading and then the cylinders were broken to determine compressive strength and modulus of elasticity.

## TEST RESULTS

### Relation of plastic flow* and modulus of resistance† to period of loading

Fig. 2 shows that for specimens of a given water-cement ratio and the same cement which were loaded to a fixed percentage of their ultimate strength, the hand-rodded specimens exhibited greater plastic flow than the vibrated specimens. In general, the cylinders made with cement 4 exhibited plastic flow for a much longer period than those made with cements 1 and 2. Table 4 gives comparative values of instantaneous deformation, plastic flow, shrinkage, and recovery upon unloading.

Fig. 3 indicates the effect of plastic flow in a somewhat different manner. It shows that the modulus of resistance decreases as a straight line function of the logarithm of the period of sustained loading. This relationship held true for approximately 200 days of loading for concrete

*Plastic flow is here defined as the axial deformation due to constant sustained load. It is the difference between the total deformation at any age, corrected for shrinkage and changes in temperature and humidity, and the instantaneous deformation which accompanies loading.

†Modulus of resistance is here defined as the ratio of unit sustained stress to unit deformation, including plastic flow and instantaneous deformation.

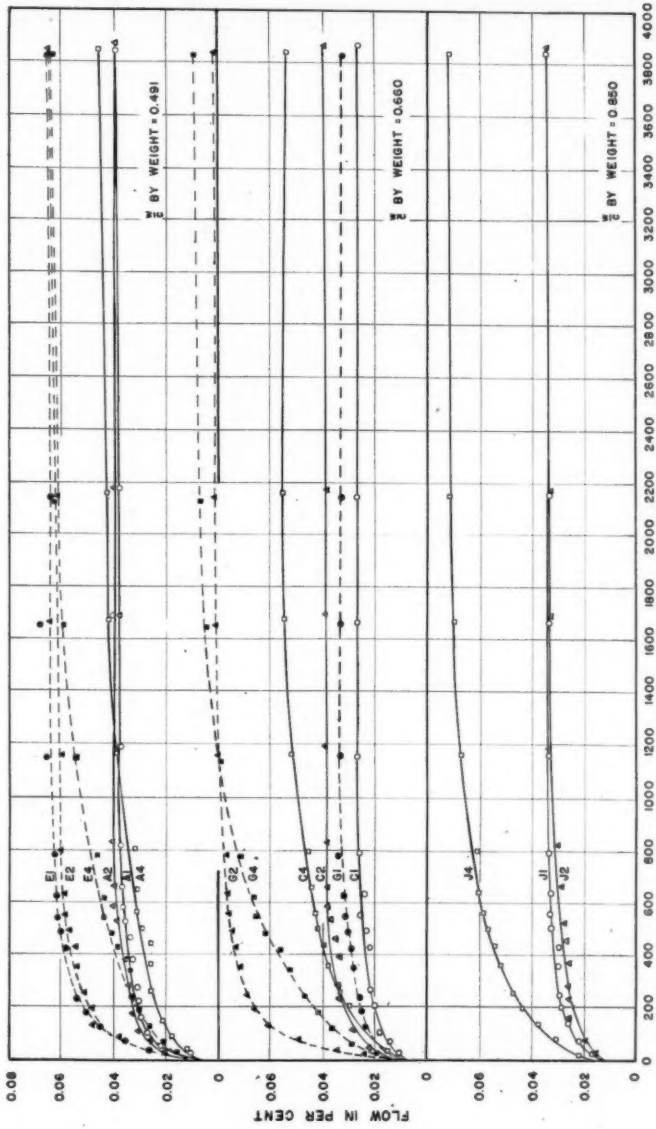


Fig. 2—Plastic flow curves

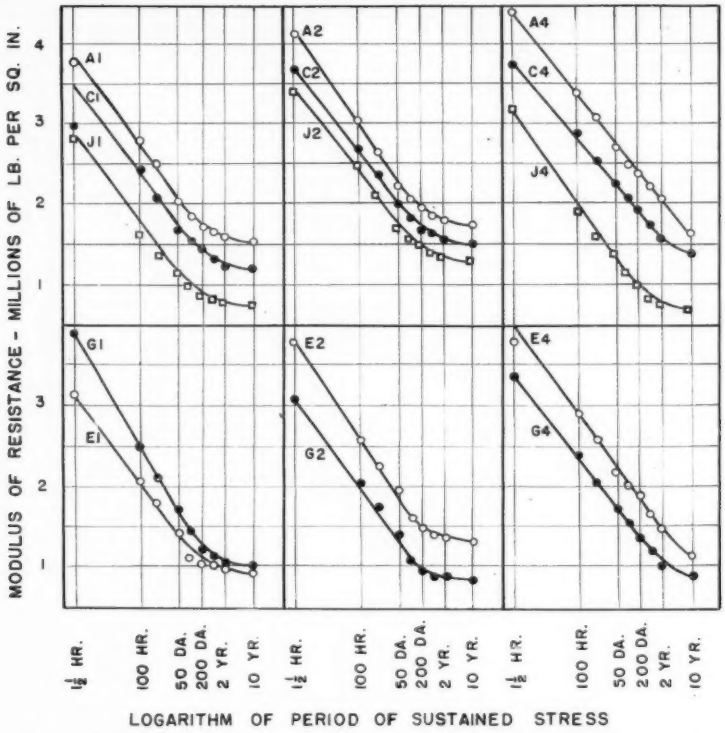


Fig. 3—Relation of modulus of resistance to logarithm of period of sustained stress

made with cements 1 and 2, and for longer periods of loading for concrete made with cement 4.

Measurements of the cylinders showed no significant change of length or recovery during the 6-month period from the time the cylinders were taken out of the loading frames until they were tested.

#### Effect of sustained loading on compressive strength

Table 5 gives the compressive strength values for specimens after 10½ years of sustained loading and for the companion specimens which were not loaded. The compressive strength of the concrete was not greatly influenced by the sustained loading. The hand-placed cylinders which had been subjected to sustained loading averaged about 5 percent stronger than the companion specimens which had not been loaded. The vibrated cylinders which had been subjected to sustained loading

## TABLE 4—DEFORMATION VALUES

| Mix | Placement | Cement | Mix prop. by wt. | W/C by wt. | Loading of plastic flow specimens | | Deformation of shrinkage and plastic flow specimens, percent | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Percent of ult. | Unit stress, psi | Instantaneous + 1½ hr. plastic flow | 2-yr. shrinkage | 10½-yr. shrinkage | 2-yr. plastic flow | 10½-yr. plastic flow | Recovery upon unloading |
| A | Vib. | 1<br>2<br>4 | 1:3:7 | .49 | 25<br>21.1<br>17.6 | 1000<br>1220<br>1220 | .027<br>.029<br>.028 | .032<br>.035<br>.040 | .030<br>.034<br>.038 | .037<br>.041<br>.032 | .039<br>.039<br>.046 | .019<br>.018<br>.020 |
| C | Vib. | 1<br>2<br>4 | 1:4½:9½ | .66 | 25<br>25<br>25 | 570<br>1120<br>1210 | .019<br>.033<br>.032 | .032<br>.035<br>.037 | .028<br>.034<br>.038 | .026<br>.039<br>.046 | .027<br>.040<br>.055 | .011<br>.020<br>.021 |
| J | Vib. | 1<br>2<br>4 | 1:6:12 | .85 | 25<br>25<br>25 | 380<br>660<br>640 | .013<br>.018<br>.020 | .029<br>.028<br>.035 | .027<br>.027<br>.035 | .034<br>.030<br>.061 | .035<br>.035<br>.072 | .011<br>.018<br>.016 |
| E | Hand | 1<br>2<br>4 | 1:2:4½ | .49 | 25<br>21.7<br>25 | 830<br>1220<br>1160 | .026<br>.032<br>.036 | .040<br>.045<br>.056 | .037<br>.044<br>.060 | .062<br>.060<br>.047 | .066<br>.066<br>.064 | .017<br>.022<br>.024 |
| G | Hand | 1<br>2<br>4 | 1:3½:6 | .66 | 25<br>25<br>25 | 450<br>940<br>1030 | .012<br>.030<br>.031 | .040<br>.048<br>.050 | .041<br>.046<br>.050 | .034<br>.077<br>.072 | .033<br>.082<br>.090 | .009<br>.018<br>.020 |

## TABLE 5—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY VALUES

| Mix | Placement | Cement | Mix prop. by wt. | W/C by wt. | Loading of plastic flow specimens | | Secant modulus of elasticity at 0.25 $f_c$ million psi | | | | Compressive strength, psi | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Control spec. | | | | Control spec. | | | |
| | | | | | Percent of ult. | Unit stress, psi | 28-day | 10½-yr. (A) | Plastic flow spec. 10½-yr. (B) | Ratio B/A | 28-day | 10½-yr. (C) | Plastic flow spec. 10½-yr. (D) | Ratio D/C |
| A | Vib. | 1<br>2<br>4 | 1:3:7 | .49 | 25<br>21.1<br>17.6 | 1000<br>1220<br>1220 | 4.34<br>4.54<br>4.54 | 5.78<br>5.87<br>5.14 | 6.53<br>6.82<br>6.07 | 1.13<br>1.16<br>1.18 | 3990<br>5820<br>6950 | 6320<br>8340<br>7080 | 5660<br>7370<br>6570 | .90<br>.88<br>.93 |
| C | Vib. | 1<br>2<br>4 | 1:4½:9½ | .66 | 25<br>25<br>25 | 570<br>1120<br>1210 | 4.34<br>4.20<br>4.56 | 5.10<br>5.40<br>5.03 | 5.94<br>6.26<br>5.98 | 1.17<br>1.16<br>1.19 | 2280<br>4470<br>4840 | 4160<br>5920<br>6150 | 4300<br>5480<br>5660 | 1.04<br>.93<br>.90 |
| J | Vib. | 1<br>2<br>4 | 1:6:12 | .85 | 25<br>25<br>25 | 380<br>660<br>640 | 4.16<br>4.42<br>4.51 | 4.09<br>4.94<br>3.67 | 4.91<br>6.02<br>4.66 | 1.20<br>1.22<br>1.27 | 1530<br>2630<br>2580 | 2230<br>3560<br>3200 | 2220<br>3470<br>3260 | 1.00<br>.97<br>1.02 |
| E | Hand | 1<br>2<br>4 | 1:2:4½ | .49 | 25<br>21.7<br>25 | 830<br>1220<br>1160 | 3.91<br>4.36<br>4.03 | 4.62<br>4.57<br>3.70 | 5.52<br>5.51<br>5.06 | 1.20<br>1.21<br>1.37 | 3310<br>5630<br>4640 | 4450<br>6000<br>4430 | 4790<br>5940<br>4560 | 1.05<br>.99<br>1.03 |
| G | Hand | 1<br>2<br>4 | 1:3½:6 | .66 | 25<br>25<br>25 | 450<br>940<br>1030 | 4.05<br>3.87<br>4.04 | 4.14<br>4.12<br>3.60 | 5.30<br>5.24<br>4.88 | 1.28<br>1.27<br>1.36 | 1820<br>3750<br>4130 | 3190<br>4200<br>4010 | 3570<br>4470<br>4380 | 1.12<br>1.06<br>1.09 |

averaged about 5 percent weaker than the companion specimens which had not been loaded.

#### Effect of sustained loading on modulus of elasticity

Table 5 also gives the modulus of elasticity values for specimens after 10½ years of sustained loading and for the companion specimens which were not loaded. The vibrated specimens which had been subjected to sustained loading showed modulus of elasticity values ranging from 13 to 27 percent higher than the companion specimens. The corresponding range for the hand-placed cylinders was 20 to 37 percent.

The specimens made with cement 4 showed greater gains in modulus of elasticity due to sustained loading than the specimens made with cements 1 and 2. In this connection, it should be noted that the companion (nonloaded) specimens of J, E, and G mixes made with cement 4 exhibited a decrease in modulus of elasticity from 28 days to 10½ years.

## CONCLUSIONS

1. The concrete made with normal or high early strength portland cement showed continued plastic flow for about two years of loading. The concrete made with high-silica cement showed continued plastic flow for about five years, and its value was usually greater than that for corresponding concrete made with normal or high early strength cement.

2. At a given age the hand-rodded cylinders experienced more plastic flow than the vibrated cylinders made with the same water-cement ratio.

3. The cylinders showed no significant length change or recovery during the 6-month period between the end of sustained loading and final testing.

4. The compressive strength of cylinders determined after 10½ years of sustained loading was about 5 percent lower for vibrated concrete, and about 5 percent higher for hand-rodded concrete, than that of companion unloaded cylinders.

5. The modulus of elasticity values determined from tests of cylinders that had been loaded for 10½ years were higher in all cases than those for the companion unloaded cylinders. The gain was about 30 percent for the hand-rodded cylinders and about 20 percent for the vibrated cylinders. The gain was greater for the concrete made with high-silica cement than for the concrete made with normal or high early strength cement.

Once a nuisance byproduct of coal-burning power plants, fly ash as a replacement for part of the portland cement in mass concrete makes possible a significant savings in material costs.

# Fly Ash as a Pozzolan*

By ROBERT F. BLANKS†

## SYNOPSIS

Fly ash is being used in combination with portland cement in the concrete mix at Hungry Horse Dam because of the benefits derived from its use, including lower cost. The effects of the fly ash being used at Hungry Horse Dam as a constituent in concrete on the workability, compressive strength, durability, permeability, heat of hydration, volume change, and counteraction of expansive reaction between aggregates and alkalies in cement are discussed briefly.

## INTRODUCTION

Fly ash is fine material consisting predominantly of small spheres of glass (Fig. 1) recovered from the stacks of certain powdered-coal-burning furnaces, such as are used in steam-electric power plants. The fineness and carbon content of fly ashes from plant to plant may vary greatly and it has been found that very fine fly ashes of low carbon content are superior when used as pozzolans. It has been demonstrated by laboratory investigations that most fly ashes are of considerable value when used in concrete because the silica glass of which it is largely composed will, in the presence of water, react with free lime, liberated during hydration of portland cement, to form a more stable cementing medium. An extensive investigation in progress for Hungry Horse Dam with a fly ash produced in Illinois, for example, indicates that a combination of approximately 30 percent fly ash and 70 percent portland cement by weight results in concrete having qualities superior to that obtained from portland cement alone. The low cost of the fly ash for this project makes it attractive as a replacement for cement even though it must be shipped long distances.

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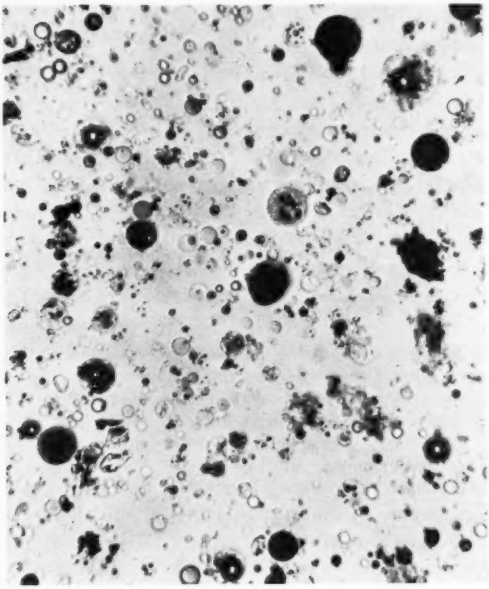

Fig. 1—Microphotograph of a typical fly ash showing the large percentage of spherical glass particles

## DISCUSSION OF TEST RESULTS

### Workability

The amount of mixing water required to produce a concrete mix having a given degree of workability is slightly less for the fly ash combination than for straight portland cement. In this respect fly ash differs from most pozzolans which usually increase the water requirement of concrete mixes. Laboratory concrete mixes containing as high as 50 percent fly ash and 50 percent portland cement by weight showed no increase in water requirements over concrete containing 100 percent portland cement. Like many other fine pozzolanic materials, fly ash improves the workability of the concrete mixes by making the mix more plastic and by decreasing segregation and bleeding.

### Strength

Fly ash replacing a portion of the portland cement is an effective strength-producing material. Tests indicate, however, that the strength gained from fly ash is not as rapid as that from portland cement. Fig. 2 shows the strength gain of 18 x 36-in. field-cycle-cured* cylinders made from concrete mixes containing Hungry Horse Dam aggregate and 3 sacks of portland cement and other mixes containing 3 sacks of cement-

*In sealed containers following the estimated field temperature cycle of the interior concrete.

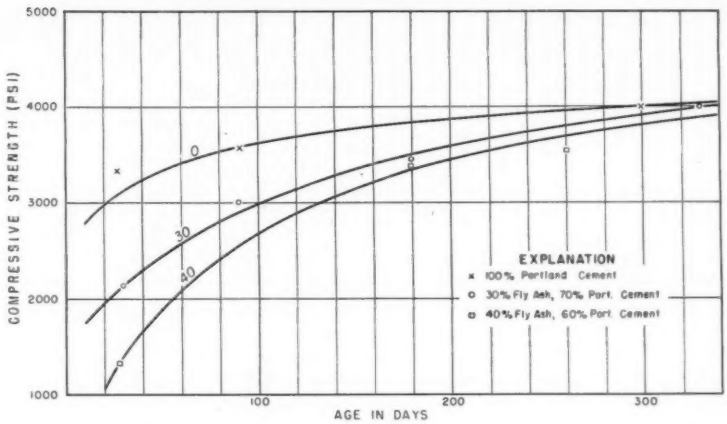


**Fig. 2—Concrete containing fly ash gains much slower at early ages but at the age of 1 year approximately equals the strength of concrete containing portland cement alone.**

ing material composed of 30 percent fly ash and 70 percent portland cement and 40 percent fly ash and 60 percent portland cement by weight, respectively. These data indicate lower strengths at 7 to 28 days for concrete containing 30 percent fly ash, but at 1-year age the strength for concrete containing fly ash approximately equals the strength of concrete made from portland cement alone. It is generally agreed in the literature that, at 1 year, the strength of concrete containing up to 50 percent fly ash and 50 percent portland cement by weight, in the majority of the cases is equal to concrete containing 100 percent portland cement.

### Durability

Tests made with Hungry Horse aggregate, a modified cement, approximately 3 percent entrained air and various amounts of fly ash indicate that the durability of the concrete is proportional to the water to cement plus pozzolan ratio, and is decreased as the percentage of fly ash is increased. These tests were made at comparatively early ages. Tests reported by others show that at later ages, on the average, concrete was 1.76 times as resistant with 20 percent fly ash by weight of portland cement. Excellent resistance to freezing and thawing, even at early ages, is developed by Hungry Horse concrete containing 30 percent fly ash, with entrained air and a reasonably low water to cement plus pozzolan ratio.

On the basis of the ratio of the compressive strength of specimens submerged in a 10 percent sodium sulfate solution for 6 months, concrete

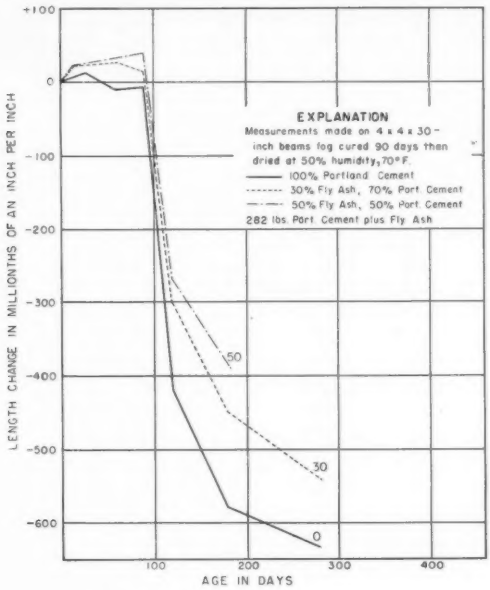


**Fig. 3—Portland cement produces greater drying shrinkage than an equal weight of fly ash.**

containing fly ash shows a superior resistance to the attack of sodium sulfate than does corresponding portland cement concrete.[2,3]

### Permeability

The permeability of concrete is an important consideration in concrete dams. Incomplete test results from the above-mentioned investigation for Hungry Horse Dam indicate that concrete is less permeable when a portion of the portland cement is replaced with fly ash than when portland cement is used alone. This indication is in agreement with the findings of other investigators.[4]

### Heat of hydration

The combination of fly ash and portland cement produces a lower heat of hydration and a consequent lower temperature rise than an equal amount of portland cement without fly ash. From the Hungry Horse mixes it has been determined that when combined with portland cement, fly ash will produce about 40 to 50 percent as much heat as an equal weight of modified portland cement.

### Volume change

Concretes containing most natural pozzolans tend to shrink more upon drying than do concretes of the same mix containing only portland

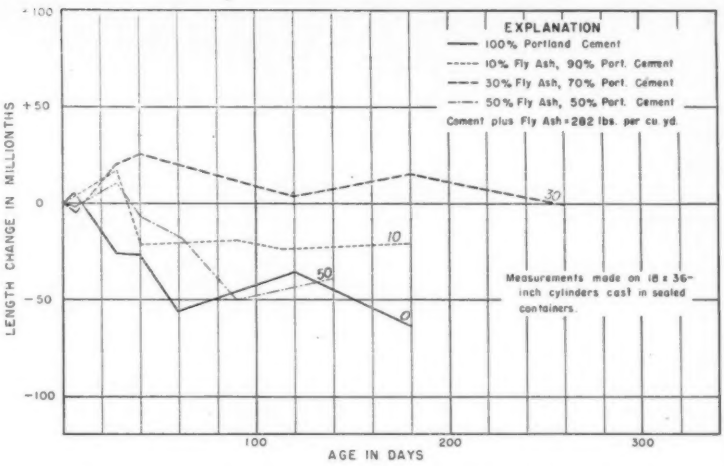


**Fig. 4—The replacement of a portion of the portland cement with fly ash does not increase the autogenous shrinkage of concrete.**

cement. Tests show, however, (Fig. 3) that for concrete containing variable amounts of fly ash, drying shrinkage is generally less than for portland cement concrete, and indicate that, on the whole, the effect of fly ash on the drying shrinkage of concrete is beneficial.

Some fine pozzolans are believed to contribute to autogenous* shrinkage. Observed values of autogenous volume change of concrete containing Hungry Horse aggregate, Type II modified cement, and variable amounts of fly ash are plotted in Fig. 4. These data show a generally lower autogenous shrinkage in concrete containing fly ash although portland cement alone shows only a very slight amount of this type of volume change. As a result of these and comparable tests, autogenous shrinkage has not been given consideration in the design of Hungry Horse Dam.

#### Alkali-aggregate reaction

Most fly ashes are effective in reducing mortar bar expansion caused by alkali-aggregate reaction. Test results[5] indicate that samples of calcined shale and raw opal are more effective than fly ash in reducing alkali-aggregate reaction, but fly ash is more effective than raw shale, slag, and various other pozzolans. Some pumicites and other volcanic ashes are less effective while others are more effective than various fly ashes in reducing alkali expansion. The test results shown in Table 1 for

*"Reaction within itself." Autogenous shrinkage is not due to any external influence but is caused by continual reaction of ingredients in sealed containers after the concrete has hardened.

**TABLE 1—EFFECTIVENESS OF FLY ASH FOR CONTROLLING ALKALI EXPANSION—PYREX GLASS MORTAR BAR TEST***

| Source of fly ash | Lab No. | Percent of cement replaced by wt. | Percent reduction in expansion | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 6-day | 14-day | 21-day | 28-day | 2-month | 4-month | 6-month | 12-month | 18-month |
| Unprocessed | 3327 | 10 | 27 | 42 | 43 | 43 | 40 | 38 | 37 | 36 | 36 |
| | | 20 | 56 | 65 | 68 | 69 | 68 | 67 | 66 | 65 | 65 |
| | | 30 | 76 | 82 | 84 | 85 | 85 | 84 | 82 | 81 | 81 |
| | | 50 | 88 | 93 | 94 | 95 | 95 | 96 | 95 | 95 | 95 |
| State line, Chicago | 8511 | 10 | 28 | 20 | 34 | 38 | 43 | 43 | 41 | 44 | 42 |
| | | 20 | 30 | 36 | 45 | 53 | 60 | 66 | 67 | 69 | 68 |
| | | 30 | 53 | 57 | 63 | 69 | 74 | 78 | 78 | 79 | 79 |
| | | 50 | 67 | 72 | 76 | 80 | 84 | 87 | 87 | 88 | 88 |
| Fisk Station, Chicago Station 1, Unit 17 | 8728 | 10 | 24 | 40 | 48 | 53 | 58 | 60 | 60 | 59 | 59 |
| | | 20 | 51 | 62 | 68 | 72 | 77 | 79 | 80 | 79 | 78 |
| | | 30 | 60 | 70 | 75 | 79 | 81 | 85 | 86 | 85 | 85 |
| | | 50 | 84 | 90 | 92 | 93 | 94 | 95 | 95 | 95 | 95 |
| Calumet Station, Chicago Station 5, Unit 7 | 8729 | 10 | 20 | 36 | 46 | 52 | 57 | 57 | 57 | 55 | 55 |
| | | 20 | 33 | 46 | 57 | 63 | 71 | 74 | 75 | 74 | 74 |
| | | 30 | 45 | 57 | 66 | 71 | 77 | 80 | 82 | 81 | 80 |
| | | 50 | 76 | 82 | 87 | 89 | 91 | 92 | 93 | 92 | 92 |
| Northwest Station, Chicago | 8730 | 10 | 27 | 40 | 50 | 55 | 62 | 63 | 64 | 63 | 62 |
| | | 20 | 45 | 56 | 64 | 70 | 75 | 77 | 77 | 76 | 75 |
| | | 30 | 56 | 68 | 75 | 78 | 83 | 84 | 85 | 85 | 85 |
| | | 50 | 79 | 86 | 89 | 91 | 93 | 93 | 94 | 93 | 93 |
| Trenton, Mich., (Detroit Edison Co.) | 8735 | 10 | 26 | 32 | 39 | 45 | 52 | 47 | 45 | 45 | 45 |
| | | 20 | 33 | 37 | 46 | 53 | 60 | 62 | 62 | 62 | 62 |
| | | 30 | 36 | 43 | 52 | 59 | 65 | 68 | 69 | 69 | 69 |
| | | 50 | 75 | 80 | 84 | 86 | 89 | 89 | 89 | 89 | 89 |

*1:2.25 mortar. Stored moist at 100 F.

various samples of fly ash indicate that fly ash is more effective in reducing expansion at later ages than at earlier ages.

## PLANT BLENDED VERSUS SEPARATE BATCHING OF FLY ASH

About 87 percent of the cost of fly ash used at Hungry Horse Dam is for shipping charges. In spite of this expense, fly ash is delivered at approximately one half the cost of portland cement. This favorable economic status becomes very attractive when large quantities of concrete are involved and represents a saving of approximately $1,678,600 in the cost of cementing materials for Hungry Horse Dam.

This saving is offset to some extent by the cost of handling an extra material and less efficient blending and mixing. A blended portland-pozzolan cement delivered to the project would be preferable to batching portland cement and pozzolan separately at the job because of improved uniformity in concrete mixing and control.

The concrete mix data (laboratory) for both interior and exterior (face) concrete of Hungry Horse Dam are shown in the Table 2.

To date some 63,000 cu yd of concrete have been placed at Hungry Horse Dam, with no unusual difficulties or experiences other than those normally encountered in the job initiation and plant adjustment stages

**TABLE 2—CONCRETE MIX DATA**

| | Interior concrete | Exterior (face) concrete |
|---|---|---|
| Portland cement, lb per cu yd | 188 (68%) | 282 (75.8%) |
| Fly ash, lb per cu yd | 90 (32%) | 90 (24.2%) |
| Aggregate,* lb per cu yd | 3712 | 3587 |
| Water, lb per cu yd | 129 | 146 |
| W/C + P ratio, by wt. | 0.46 | 0.39 |
| Sand, percent | 23 | 22 |
| Entrained air, percent | 3 ± 1 | 3 ± 1 |
| Slump, in. | > 2 | > 2 |
| Compressive strength at 1 year, psi | | |
| 18 x 36-in. sealed cylinders | 4000 | 4500 |
| 6 x 12-in., standard fog cure | 5000 | 6200 |

*6-in. maximum size.

of a large project. At some future date a full report on the job experiences and results with fly ash concrete on this project will be compiled.

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Title No. 46-52

Another viewpoint on inspection. This on the personal attributes of the inspector which make for success.

# The Inspector*

By MILES N. CLAIR†


## SYNOPSIS

The functions of the inspector include cooperation with the contractor in planning for good workmanship and progress, observation of construction operations and the recording of important facts relative to the work, and the enforcement of specifications and good construction practice. The qualifications for the inspector are discussed; he must be experienced, interested, be able to cooperate, be honest and have lots of common sense.


The concrete inspector, in common with all other inspectors, is looked upon by most contractors and contractor's personnel as one of the necessary evils, at best, related to construction and at the worst as a nuisance and a principal obstacle to progress. The reason for this unfortunate position is to be found partly in the work the inspector has to perform, his characteristics, the actions of his superiors and the attitude of the contractor. The work that inspectors must perform to insure compliance with the plans and specifications tends to cause a conflict with the contractor. No one likes to have his mistakes brought to his attention. The contractor, even if sincere and competent will, because of the lack of perfection common to all humans, not perform some operation properly. He will know that this is so but perversely will argue about the matter if it is drawn to his attention for correction. A good inspector will know that this is true and will give ample time for the contractor to find and correct his own mistakes.

## THE INSPECTOR

A large part of the difficulties an inspector encounters is due to his own characteristics. A necessary quality in any one dealing with contractors is a good measure of common sense. An inspector will not

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gain the respect of the workmen or of the contractor if he measures slump in eighths of an inch or goes into a tirade about "lousy aggregates" where the deviation is 1 percent from the allowable gradation on one sieve size. He will not be very happy on a job where the slab finish has been spoiled by rain and his sole contribution is an assertion to the effect that "any damn fool could have known that it would rain before the work was completed" when he was a party to the whole procedure and deserves the appellation if any one does.

The inspector will gain the respect of all parties if he recommends rain frames or other protection and where necessary gives the details of the same and insists that they be available at the place where needed and that they are used as needed. There is nothing accomplished by damning the contractor for the honeycomb that shows up when the forms are stripped if the inspector did not reject the buggy load of concrete that was the cause or if he stood by and said nothing while spading was neglected or if he allowed placement to proceed when there was inadequate personnel for placement. There is no use in berating the carpenters for poor formwork when the stripped concrete shows voids and fins due to open joints. The inspector must see that the forms are proper before the concrete placement is allowed to start. Any one knows that practically nothing satisfactory can be done about a reinforcing rod that shows on the surface after the concrete has hardened but a lot can be done by a good inspector before and during the placement of concrete.

A competent inspector will not shut down a placement because 2-in. coarse aggregate is temporarily not available while a 1-in. size can be obtained to take care of the emergency. He will, however, take steps to avoid a recurrence of the situation by investigating the cause and have the contractor take the necessary precautions.

The inspector can obtain cooperation from the contractor by friendly discussion of the progress of the work in the course of which he brings out the items that must be ordered ahead or work to be done in preparation.

The coming of cold weather may be, under one interpretation, an act of God, but is not as unexpected as a tornado and comes each winter, therefore an inspector who has not thoroughly reviewed the necessary procedure and agreed on details with the contractor before concreting is allowed to proceed during the normally cold season has no one but himself to blame for frozen concrete.

## THE CONTRACTOR

The inspector must start on the premise that the contractor wants to do a good job at the least possible cost. There are, however, con-

tractors who do not know how to do proper work and there are those who apparently do not want to do a good job. They all present the same problem but with varying difficulty. The competent contractor who wants to do a good job normally offers no problems to the inspector except that of interpretation of plans and specifications. The willing but incompetent contractor can be guided by an experienced inspector so as to get a reasonable job. The inspector under these conditions is forced to function unofficially as a part of the contractor's organization. The best solution of the problem is to recognize the difficulty at the start of the work and demand the employment of proper personnel by the contractor.

The contractor who may or may not be competent but who is not particularly interested in doing a good job is a real problem for even the best qualified inspector. Such contractors are most likely to be encountered under the system of competitive bidding required for public work and where political considerations may affect the award irrespective of the competency of the contractor. The inspector may have to utilize every "club" provided by the contract to insure that good workmanship is obtained but he must not be baited into an attitude of antagonism or become involved in personalities. Great care must be taken to keep full records on all instructions given and agreements made when dealing with this type of contractor.

The inspector should take particular care to start the job right by insisting on good workmanship. The rejection of a buggy or load of concrete, or the refinishing of a slab, the resetting of steel or rebuilding of forms teaches a lesson that is not soon forgotten. The requested removal of contractor's personnel that will not cooperate also solves many problems.

## RELATIONS WITH SUPERIORS

The inspector is in an impossible position unless he has the full confidence and backing of his superiors. To maintain this condition the inspector must keep his superiors fully informed by daily written and oral reports and when in doubt or where a controversy may arise he should discuss the matter with them. Two heads are normally better than one and a person detached from the details may offer a solution that is not readily apparent at the site. Despite every effort, personalities become a factor in the dealings between the inspector and the contractor and this factor can be largely eliminated in a decision made by some one other than the inspector. The inspector's superior must render decisions promptly and clearly. He also should cause immediate compliance by the contractor with the requests of the inspector. Particularly necessary is action to stop any badgering of the inspector by the con-

tractor or his employees. Also, as inspectors are human, despite some references to the contrary, they need indications of appreciation by their superior for a job well done.

## SUMMARY

The inspector must see that the plans and specifications are fulfilled and translated into good construction. He must attempt to be a co-ordinator, expediter and adviser in addition to a checker. To do all these things he must have knowledge and experience, the ability to get along with and deal with men, the ability to make a decision and stay with it. In addition, he must have a real liking for his work and the kind of interest that makes him go in on holidays to see if the curing is really being maintained. He must not do this as a detective to "catch" the contractor at something but because he realizes that on such days the man on duty often gets careless.

What kind of a man will meet these requirements? No man will fully meet them but the best men are graduate engineers with knowledge of concrete mixtures and structural design and with five or more years of field experience, and who are honest, alert and not afraid of getting dirty by actually showing a workman how to spade, use a vibrator or move a chute.

With the recognition of the importance of improvement in inspectors there will come great improvement in concrete.

A Code governing prestressed concrete? Not now, says the author.

# Patents and Codes Relating to Prestressed Concrete*

By CURZON DOBELL†


## SYNOPSIS

Based on a study of more than 100 domestic and foreign patents and on correspondence with committees of engineers in England, Switzerland, France and Sweden this paper gives a brief account of the more important groups of patents, a synopsis of codes in preparation in the above countries and the problems to be overcome in drafting a prestressed code for domestic use.


## INTRODUCTION

So much has been written about prestressed concrete it has become almost impossible to find an original topic for discussion. To date 43 books and more than 500 articles have been published; almost as many as the number of prestressed concrete structures which have been built in the world up to this time. For this reason this paper will be confined to patents and codes, which have received only passing reference in the voluminous literature on prestressed concrete.

## PATENTS

If we accept as a definition of the principle of prestressing "The action upon a structure of artificial external forces calculated to produce stresses of sufficient magnitude to permanently neutralize undesirable stresses of opposite-sign due to load" it is clear that no inventor can claim a basic patent on the principle of prestressing, for it concerns only the natural laws of mechanics which cannot be invented by man. In fact, the principle of prestressing, under our definition, has been employed to improve the stability of structures from the earliest times.

Every farmer who has placed a tensioned hog-rod under a sagging beam to prevent its collapse has unwittingly used the principle. The

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first man to conceive of curving the staves of a barrel so as to force metal hoops around them from opposite ends, was probably the first to apply the principle to a circular structure. Indeed, the first woman to wear a corset was, whether she knew it or not, likewise applying external forces to neutralize undesirable bursting stresses and improve the stability, as well as the appearance, of the structure.

#### Early developments

Probably the first scientific use of this principle, upon which patents were granted, was the wrapping of gun barrels with tensioned wire to place the barrel in compression so that the force of the detonation and passage of the projectile through the barrel would not stress the metal beyond the yield point and thus produce a permanent enlargement of the bore. This use dates back to the Civil War when U. S. Patent No. 34,977 was granted to *Ostrander* in 1882, followed by patent No. 568,914 to *Miller* in 1896, patent 689,413 to *Roberge* in 1901 and patent 734,965 to *Schinneler* in 1903, all for the same use. A similar patent, but for the wire wrapping of wood stave pipe, is No. 1,006,173 granted to *Aine* in 1911.

In considering the early patents for applying the principle of prestressing to masonry structures, it is interesting to note that, contrary to popular misconception, many of the early inventors were American. Authorities on the subject agree that the first such patent in any country is U. S. Patent No. 375,999 applied for in 1886 and finally granted in 1898 to *P. H. Jackson* of San Francisco. This patent, entitled "Constructions of Artificial Stone and Concrete Pavements," describes several methods of stretching ties between the footings of arches by skewbacks, turnbuckles, wedges, etc., and clearly falls within our definition.

In 1888 a patent was applied for in Germany by *C. F. W. Doehring*, but we have no record of any corresponding U. S. application. In this, triangular members for use in strengthening timber floors were manufactured from mortar and tensioned wire.

There are some confused references to German patent applications by *J. Mahal* in 1896 to "utilize to the maximum the strength of the concrete by reducing the concrete tensile stresses under load" and by *M. Koenen* in 1907 for a "formula to determine the stretching force necessary to limit to a certain magnitude the triangular straight line stress distribution obtained from the working load."

After Jackson, in 1886, the next U. S. Patent on record was No. 778,416 granted in 1904 to *R. C. Kyle*, which described a method for prefabricating the reinforcing for a beam. The reinforcing was made rigid and self-supporting by a gridwork of tensioned cables reacting

against end plates spread by steel channels. The entire reinforcing assembly was then placed in a mold to receive concrete.

In 1907, U. S. Patent No. 1,020,587 was granted to a Norwegian, *J. G. F. Lund*, for the manufacture of vaults, consisting of rows of "brittle" blocks, jointed in mortar, with prestressed tie rods arranged between the rows in a mortar joint with the compression transmitted to the blocks by nuts and washers at the ends of the rods. In 1908 patent No. 903,906 was granted to an American, *G. R. Steiner*, in which he proposed to tighten the reinforcing rods first against green concrete to destroy the bond and then increase the tension after hardening of the concrete.

It is clear from these early patents granted in the ten-year period from 1898-1908 that the inventors had a clear concept of improving the stability of masonry structures by the use of prestressing, but the meager records of the period show that the early experiments failed to support the theory upon which they were based, leading to the abandonment of any further attempts at prestressing for the next twenty years. We now know that these early failures resulted from a lack of comprehension of the phenomenon of plastic deformation in concrete, a thorough understanding of which is a prerequisite to successful prestress design. Time does not permit a discussion of this all-important phenomenon which has been dealt with in detail by authorities such as Freyssinet, Magnel, Schorer and Crom. However, it may be noted in passing that this same lack of comprehension persists even today in the acceptance by some engineers of steel stresses so low that they cannot absorb losses of stress due to shrinkage and plastic flow in the concrete and still leave sufficient residual stress to maintain the concrete in permanent compression under load. This dangerous condition is most often encountered in tanks stressed by rods tightened with turnbuckles.

#### Recent patents

In tracing the history of patents on prestressed concrete, we must jump 20 years from 1908 to 1928, during which time, with one exception, we have no record of any technical writings or patents on the subject. This exception is a Viennese, *K. Wettstein*, who has recently come forward in a letter to the Swiss Assn. of Material Testing stating that as early as 1919 he produced in Austria prestressed concrete planks reinforced with highly tensioned piano wire and citing tests at the Technical University, Vienna, in 1924, thus seeking priority over Freyssinet and Hoyer. However, no patents appear to have been granted to Mr. Wettstein and substantial further evidence is needed to grant him such a place of honor as the originator of the modern concept and technique of prestressed concrete.

We now come to Freyssinet, Dill, Hoyer, Hewett, Crom, Abeles, Magnel, Blaton, Lossier, etc., well-known in the modern development of prestressed concrete. It is difficult to decide who was the first to perceive the importance of plastic deformation, and to fully measure its varying magnitude and account properly for it in design, but most authorities agree the credit should go to Freyssinet.

*Eugene Freyssinet*, a French engineer, has devoted the past 20 years to thorough research in this field. He has written profusely on the subject and his last lecture, delivered in England on November 17, 1949, before a joint meeting of the Institution of Civil Engineers and Societe des Ingenieurs Civils de France, should be read and thoroughly understood by any engineer desirous of entering this field. It is so scientific as to be philosophical. Mr. Freyssinet's first patent shows an appreciation of the necessity of taking plastic flow into consideration, but it lay dormant for many years under the innocuous title of "Piece of Reinforced Concrete," in contrast to high-sounding titles bestowed on other insignificant patents. This patent was first filed in France on October 2, 1928, maturing into French Patent No. 686,547 on January 22, 1930. A corresponding U. S. application was filed in 1934, which finally issued as patent No. 2,080,074 on May 11, 1937. The first claim of this patent is as follows:

"A piece of reinforced concrete which comprises set and hardened concrete having undergone shrinkage, and reinforcements of a steel having a high elastic limit in permanent tension anchored in said concrete and adhering thereto along their whole length, whereby said concrete is permanently subjected to compressive stress."

Since then, Mr. Freyssinet's labors have been rewarded by some ten subsequent patents covering his methods in beams and pipes.

Some authors have credited *R. E. Dill* of Alexandria, Neb., as the first to understand the need for using high steel stresses in prestressed concrete, but his patent No. 1,686,633 issued September 18, 1928 makes no such claim. It claims the method of coating threaded rods to prevent bond, so the rods may be tightened with nuts and washers after the concrete has hardened. Large quantities of posts were built under this patent in the western states and the same patent, under license, was later used by the Preload Co. in the construction of its first prestressed tanks in 1935. The same idea of preventing bond of reinforcement, later to be stressed against the concrete, was the basis of patent No. 1,781,699 to *Parmely* in 1930 and patent No. 2,185,749 to *R. D. Kennedy* in 1940, since assigned to the Preload Corp.

In 1938, *Edwald Hoyer*, developed a method in Germany of stretching large numbers of fine piano wires to extremely high stresses in external frames so that concrete could be poured around the wires which were later released after the concrete had set. Anchorage was solely de-

pendent on bond so that concrete beams and slabs built by this means could be cut later to desired lengths. Patents on this method were granted in most countries, including Canada, but were denied in the United States. By this method small beams and slabs were produced in large quantities in Germany during the war, and new factories have since gone into production, particularly in the Scandinavian countries.

The first to apply the principle of prestressing to circular tanks was *William S. Hewett* of Chicago, who, in 1937, was granted patent No. 2,091,444, for stressing rods by means of turnbuckles around circular walls, later protected by a covering of concrete and Gunite. *J. M. Crom*, who was then associated with Hewett, realized the impossibility of creating, by this means, sufficiently high stress to obtain permanent compression in the concrete and so started experiments which led to his first patent, No. 2,364,966. This was followed by five subsequent patents for methods employing high-tensile wire in the construction of large circular tanks and domes.

In 1941, *W. B. Miller*, of the Lewiston Pipe Co. of Chicago, was granted patents 2,236,107 and 2,236,108, which describe certain techniques for wrapping pipe with high tension wire covered with Gunite for protection.

The most recent and perhaps the most practical patents granted for prestressing linear members, such as beams and slabs, are those granted in European countries to *Gustave Magnel* and *Entreprises Blaton-Aubert, S. A.*, of Brussels, and still in application form in the U. S. Patent Office. Professor Magnel's principles are fully explained in his book, "Prestressed Concrete" published in 1948. To date these patents have been employed in the construction of many large buildings, hangars and bridges in Europe, including the first continuous span bridge in prestressed concrete over the Sclayn River in Belgium. The Walnut Lane Bridge in Philadelphia, about which so much has recently been written, also employs these patents.

The last name to consider, and this from the standpoint of novelty, is that of *Henry Lossier*, who has been granted patents in Europe for the use of expansive cements to produce controlled expansion of concrete against the resistance of imbedded reinforcing bars, thus producing the desired degree of compression in the concrete. Mr. Lossier has written learnedly in *Ingenieur Civil* and other European publications on this theory, but it is hard to imagine an average American construction crew producing concrete to which must be added the precise amounts of several chemicals at precisely the right moment during mixing, under controlled atmospheric conditions to obtain exactly the right degree of elongation of a beam.

So far about 80 U. S. patents have been granted for various methods of prestressing. Of these, 56 have been granted since 1933 and still in force, and there are at least 20 applications which are likely to mature into patents in the near future. As with all new technical developments, the patent position is still expanding and will continue to do so for some years as new inventions enter the field.

## CODES

We turn now to the relationship of prestressed concrete to building codes. The purpose of prestressing masonry structures is to remove the limiting factors in conventional reinforced concrete design, resulting from tensile, shear and buckling stresses, which have heretofore precluded the use of high compression concrete and high tensile steel. Through prestressing, a homogeneous material is achieved in which, within the limits of design load, the concrete acts only in compression and the steel only in tension, and the neutral axis can be controlled to lie entirely outside the section. By so eliminating or controlling undesirable stresses in the concrete there is no longer any design limitation on the stress which may be used. In fact, from the standpoint of design the higher the stresses the better; the only limitation on stresses being the respective strengths of the concrete and steel available.

### Design considerations

The conditions which dictate code requirements are quite different, and sometimes opposed, for prestressed concrete as compared to those for standard reinforced concrete. For example, in standard reinforced concrete design it is assumed that the concrete will crack before the reinforcing can assume the tensile load, and it is therefore necessary in a code to place limits on the permissible reinforcing stresses and elongations consistent with the type of structure. In prestressed concrete there is no tension or cracking in the concrete so elongation of the reinforcing is no longer a factor. Instead, consideration must be given to the plastic flow caused by the prestress, and shrinkage in the concrete, and to a lesser degree in the steel.

With initial compression of 1500 to 2500 psi, as normally used in prestressed design, these deformations have a composite coefficient of 0.0007 to 0.0018, which, in terms of loss of steel stress, is equivalent to 20,000 to 50,000 psi based on elastic modulus of 28,000,000 psi. This loss of stress is almost constant regardless of the initial steel stress used to create the desired compression in the concrete, and must be added to the design working stress to determine the initial steel stress to insure absence of tension in the concrete. The loss of stress is affected by many variables, including age at prestressing, temperature, humidity,

shrinkage and plastic deformation, but 30,000 psi is taken as an average in the following examples.

If a low initial steel stress of about 50,000 psi is used, as in the case of high carbon rods stretched by nuts or turnbuckles, the design working stress will be 50,000 − 30,000 or only 20,000 psi which gives a ratio of initial stress to design working stress of 5 to 2. As the area of the compression concrete is governed by the *initial* steel stress and the area of steel is governed by the *design* steel stress, it follows that both the concrete compression area and steel area in this case will be 150 percent greater than would be required if it were not necessary to allow for plastic losses. However, if a high initial steel stress of about 150,000 psi is used, as is possible with specially treated cold-drawn spring steel wire, the ratio of initial stress to design working stress becomes 150,000 to 120,000 or 5 to 4; so that now the increase in concrete compression and steel areas will be only 25 percent greater than that required if it were not necessary to allow for plastic losses. High steel stresses produce correspondingly great elongations which are easier to adjust and maintain, and have the further advantage that should the actual loss of steel stress be greater than that allowed for in design, the resulting tension in the concrete will develop at a reduced rate corresponding to the reduced ratio of the loss of steel stress to the initial steel stress.

These examples lead to a paradox which is nevertheless one of the most important considerations in prestressed concrete design. In standard reinforced concrete design, safety and stability are on the side of low steel stresses, whereas in prestressed concrete the reverse is true; and the only limit on the steel stress which should be used is the physical properties of the steel itself.

Take the case where the dead load equals the live load. Here, in standard design, the steel stress becomes 100 percent greater when the live load is applied than it was under dead load alone. In the case of prestressed design, since the concrete is in compression and uncracked, the steel stress change is only the change in concrete stress multiplied by the elastic modular ratio of steel to concrete. The drop in compression in prestressed concrete usually is not in excess of 1200 psi so that the increase in steel tension stress is only about 8000 psi. Compare this 8000 with the initial stress of 150,000, and the stress change is about 5 percent as against the 100 percent in standard design. However, there is usually a period of at least several weeks before the live load comes into play during which the steel stress will drop perhaps 20,000 psi or more to conform to the shortening of the concrete. This is equivalent to 2.5 times the subsequent increase of 8000 psi in steel stress due to live load. In practice, the reinforcing in prestressed concrete is tested the day the prestress is applied to a stress of 10 percent to 20

percent greater than it will be required to sustain under actual working load.

It would therefore be absurd to require a safety factor of 2 to 2.5 times the ultimate strength in prestressed reinforcing as is properly required in most codes for standard reinforcing. In fact, at this time, most authorities agree that for bonded prestress reinforcing the initial tension may be taken as 85 percent of the yield point or 70 percent of the ultimate strength, whichever value is the lesser.

From the foregoing examples it is clear that codes designed for standard reinforced concrete cannot be applied to prestressed concrete, which is really a new homogeneous building material with high elastic properties.

**Present codes**

To date there have been two attempts to write codes for prestressed concrete.

The first, written in Polish and translated into German, is the work of a committee of about 20 Polish refugee engineers located in Switzerland during the war, which, with the cooperation of the Federal Polytechnic Institute of Zurich, wrote a tentative code for prestressed concrete for monolithic construction without end anchorages. This code is only intended for Hoyer type prestressed concrete, mentioned in the first part of this paper. It is limited to simple beam elements, of a maximum span of 18 ft. A clause in this code provides that for other structures which are not manufactured in series, special approval must be obtained from case to case if the span exceeds 18 ft.

The text of this code is significant as it shows how thoroughly the various problems of both design and manufacture were considered. The scope of this paper does not allow comment on all the various chapters of this code but it is basically a well conceived document. In fact, in many instances, it goes beyond the normal purpose of a code and is almost too restrictive. At the end of this code a considerable amount of technical data is given in the form of curves for all pertinent prestress design.

The second publication which comes within the classification of a code for prestressed concrete, rather than just a design paper, is Kurt Billig's "A Proposal for a Draft Code of Practice for Prestressed Reinforced Concrete." This paper was published in London in January 1948, and is a rather comprehensive tentative code for this new field of construction. It is, to my knowledge, the only publication of its kind in the English language.

The Billig code is subdivided into two parts, General and Design. Part I covers definition, general specifications, materials, information on shrinkage and creep and prestressing with or without bond, as well

as construction questions. Part II deals with basic design concepts, design assumptions and notations, followed by a comprehensive discussion of the permissible and limiting stresses for various loading conditions and the precompression losses due to shrinkage, creep and flow, etc. At the end of the second part of this code the questions of buckling under preliminary forces, resistance to cracking and factors of safety are discussed. This is one of the most important portions of the code, and might better have been placed at the beginning of the design section since the conception of safety factor, still a rather controversial subject among engineers, is the over-all governing criterion.

With the exception of these two publications, the only other papers which could possibly claim to be a basis for a code are the two articles by Herman Schorer in the ACI JOURNAL of September 1946 and June 1943. However, these are purely design papers and it was probably not the intention of Mr. Schorer that they be regarded as a basis for a code.

#### Is a code for prestressed concrete needed now?

A legally enforceable code is highly desirable to control any profession concerned with the public safety and welfare, such as medicine, engineering or navigation, provided the practice of the profession has reached such a degree of uniformity that it may be confined within the framework of a code without unduly curbing initiative and progress. However, a code, to be effective, must impose boundaries of safety around all aspects of its subject to prevent the venturesome or unskilled from trespassing into dangerous territory. Once a code is adopted it becomes the law, and the law must be complete as it tends to replace the broader restraints of conscience and ethical standards.

At the present stage of its development are we ready to draft a comprehensive code for prestressed concrete to include all classes of structures, design requirements and techniques of prestressing? The author thinks not. At this moment there is disagreement among the leading designers in the field about safety factors, design notations, steel stresses, plastic deformations in steel and concrete, and many other basic considerations. Until last November, for instance, everything written on the subject, including the two draft codes mentioned above, agreed with the postulates laid down by Hoyer that bond alone provides sufficient anchorage if the diameter of the reinforcing elements is below 2 mm. In his latest paper, referred to earlier, Freyssinet advances the theory that this was a fallacy and that the larger the diameter of wire the greater the anchorage achieved by bond.

To date nearly all prestressed concrete structures have followed the principles of design and methods of application developed by the four engineers mentioned earlier who spent large sums of money to

develop and test their ideas and had the prudence to protect them with patents. We cannot limit a code to these tried and tested methods, but dare we broaden it to give official sanction to hastily conceived ideas insufficiently supported by research to guarantee their equality with these established methods?

At a recent conference on prestressed concrete at the Institution of Civil Engineers in England the question of adopting a code for prestressed concrete was raised. Gustave Magnel, leading continental authority on the subject said, "For goodness sake, don't do it. Believe in freedom!" He went on to suggest that it would be no good making a Code of Practice when no routine method had been established even after some years of practice. His opinion was that if prestressed concrete structures were designed by first class brains and made by first class contractors the necessary codes could follow, but that under present conditions a code would be a drag on progress.

The whole field of prestressed concrete is still in such a fluid state that great harm might result in freezing it in its present stage of development by the imposition of an official code.

The author therefore recommends that ACI Committee 323 refrain from drafting or sponsoring any code until at least some semblance of agreement is reached among the leading authorities in this field.

Discussion of a paper by Curzon Dobell:

# Patents and Codes Relating to Prestressed Concrete*

By J. J. POLIVKA and AUTHOR

By J. J. POLIVKA†

The author states that Karl Wettstein is seeking priority over Freyssinet and Hoyer in patents on prestressed concrete and that further evidence is needed to grant him such a place of honor as the originator of the modern concept and technique of prestressed concrete. The writer was Wettstein's technical advisor and consultant in the early stage of his developments and made extensive studies and tests of his resilient concrete planks, plain and with insulating core, prestressed by high tension piano wire‡ and believes himself competent to furnish this evidence.

Recently, E. Walter, president of the Porete Manufacturing Co., North Arlington, N. J., in a letter to the editor of *Engineering News-Record* (Oct. 20, 1949, p. 100) pointed to the fact that the idea of prestressing concrete for structural purposes was conceived by Wettstein in 1919 when his method of increasing the strength and elasticity of thin precast structural members (roof and floor slabs and joists) was first applied and tested. Walter refers to Coff's article in *Engineering News-Record* (Sept. 1, 1949), especially to his statement that "bonded prestress was initiated by Freyssinet and perfected by Ewald Hoyer in Germany, and Hoyer has placed bonded prestressed concrete on an industrial basis for short spans." The writer confirmed Walter's statement regarding Wettstein's priority in the Dec. 8, 1949, *Engineering News-Record*, p. 144.

Wettstein received his first patent in Germany, valid from Jan. 27, 1921, and subsequently applications for world patents were made. The German patent contains in its claims all the characteristics of modern bonded prestressing and is a basic document for Wettstein's priority.§

*ACI JOURNAL, May 1950, *Proc.* V. 46, p. 713. Disc. 46-53 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

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‡Polivka, J. J., "Wettstein's Concrete Planks," *Teyssler-Kotyska* (Technical Encyclopedia), V. 15, Prague, 1933, pp. 335 and 336, translation in Appendix.

§ Karl Wettstein, Most (Bruex), Bohemia, "Process of Manufacturing Planks and Other Especially Flat Concrete Units with Wire Reinforcement," Patent No. 384009, Category 80 a, Group 48 (W 57537 VI 80 a¹), patented Jan. 27, 1921.

### Wettstein's prestressed concrete planks

After World War I a considerable lumber shortage existed in some European countries, especially boards, planks and small-sized joists and rafters. Wettstein, at that time a general contractor in Most, in northwestern Bohemia, conceived the idea of replacing the most critical boards and planks down to one inch thickness (20 and 25 mm = 0.8 and 1.0 in.) by thin concrete boards and slabs having thicknesses from 0.4 to 1.4 in. and having at least the same bearing capacity as lumber. He was aware that conventional reinforced concrete never could solve this problem. From the grain of lumber he conceived the idea of using thin steel wire (32 gage = 0.3 mm), available in the form of piano wire, as reinforcement. The prototype of lumber grain bonded by resin and parallel tests showing extremely high bond values of thin wires embedded in concrete with fine aggregates persuaded him to discard any anchorage of pre-tensioned reinforcement. This elimination of anchorage also permitted cutting prestressed slabs to various lengths without reducing the strength. There was another evident advantage—prestressed concrete boards possessed two-way strength when cross-reinforcing was used, a property which is present only in plywood boards. The strength of concrete was attained by using high-strength cement and appropriate fine aggregates, and in manufacturing, vibrators and various curing methods were used. In the Most factory, mechanical vibrators were installed and the curing took place in a four-track curing tunnel. Later on, from 25 to 30 full-size boards 19 x 77 in. (48 x 198 cm) were put in a steel crate and submerged in a hot water tank. From 1921-1928 the steel wire was gradually increased to 24-20 gage (0.6 to 0.8 mm) and later to 19-17 (1.0-1.2 mm). The importance of thin wires was emphasized in his German patent of 1921, and also in his business pamphlets distributed in 1923 where it stated, "Bond is increased approximately 100 percent if a wire of certain diameter is replaced by four wires of half thickness."

### Manufacturing process

In the early development, Wettstein used mechanical equipment for pretensioning 18 gage (1.2 mm) wires up to 370,000 psi. The equipment had devices for exact determination of the stress by measuring strain, and usually the wires were prestressed to intensities close to the yield point of the steel. At that time few types of high-strength steel were available and competition with other materials made it necessary to use wires of the highest economical value, *i.e.*, having the lowest cost per stress unit. Wires of lower strength, 200,000-285,000 psi, sometimes proved to be more economical than wires with an ultimate strength of 350,000 psi and more; a 1924 pamphlet described wire reinforcement having a 285,000 psi ultimate strength.

The extremely thin concrete boards had considerable flexibility. A 0.63 in. (16 mm) thick board spanning 6 ft deflected 1⅛ in. (30 mm) under dead weight alone. For this reason wires with relatively low elongation were preferred. The two-way reinforcement also was effective in reducing detrimental

deflection. No permanent deflection remained, as was emphasized in Wettstein's July 3, 1924, pamphlet "Elastic Concrete Boards."

After 1925 Wettstein started to manufacture boards with insulating cores of lightweight concrete. Relatively thin surface layers were made of high-strength cement mortar and reinforced with wire which was again prestressed in both directions by means of standard steel pipes, the outside diameter being approximately equal to the thickness of the insulating interlayer. After hardening, the pipes were removed and reused. For protection against damage during transportation the pipes were sometimes left in until after arrival at the construction site. Usually concrete with lightweight aggregates (pumice, slag) or gas concrete was employed for the core.

In 1925-1926 Wettstein built another plant in Duchcov (Dux) with mechanical equipment suitable also for manufacturing prestressed concrete joists and beams. Similar joists and beams were recently used in the design of pier No. 57 in New York, resulting in great economy.*

#### Tests

The writer's tests at the beginning of Wettstein's developments were followed by tests in other countries which confirmed the soundness of structural principles. The engineering materials laboratory of the Technical University in Vienna, Austria, made a series of tests conducted by Rinagl under the supervision of Ludwig. The final report, May 2, 1924, states that 0.63 in. thick boards were tested to failure under gradually increasing loads. The stresses at the moment of the failure were 177,000 psi in steel and 3750 psi in concrete, and the measured deflections indicated a modulus of elasticity of 4,440,000 psi. On May 8, 1924, a report on Wettstein's prestressed boards was issued by H. Becher, consulting engineer in Berlin-Charlottenburg, in which, on the basis of his own tests, basic characteristics of prestressed concrete, as they are known today, are described.

At the beginning of Wettstein's commercial production there was doubt of permanence of pre-tensioning. After many years, boards were removed from the structure and tested in this respect. In one case, a large factory in Duchcov, Czechoslovakia, built in 1927, the roof boards were subjected to tests after 17 years. The boards were 1.2 in. thick and reinforced with 24-gage wires having an ultimate strength of 370,000 psi. Mörsch's testing method revealed that the original prestressing up to 230,000 psi was practically maintained. In another case, the Skoda Works built a factory in 1931, in which 86,000 sq ft of Wettstein's prestressed roof boards were used; the management reported 13 years after erection, that the boards were in good condition without the slightest defect. The same was found true with boards used in a factory in Kladno, Bohemia, built in 1930. The roof of Wettstein's original factory in Most, built in 1922, had to be dismantled in 1942 due to mining operations. After 18 years, the roof boards showed not the slightest cracks and no sign of rusting was noticeable on extracted wires.

*_Engineering News-Record_, Jan. 26, 1950, p. 40; _Architect and Engineer_, V. 180, No. 2, Feb. 1950, p. 31.

It is regrettable that war conditions and political hardship after World War II prevented Wettstein from defending publicly the priority of his idea, especially since during the past ten years the great advantages of this new structural material have begun to be recognized and utilized in the economical design of concrete structures. His priority over Freyssinet and Hoyer is proved definitely by issued patents, the first valid from Jan. 27, 1921. Hoyer, as generally reported, developed the idea to tension long wires in 1938, by which "practical application of prestressed concrete on a large scale became possible."* The recorded facts prove, however, that this idea was originated by Wettstein. The writer believes that the following statement by Wettstein will sufficiently explain the fatal misunderstanding.

"It can be understood why Hoyer often is considered as inventor of concrete with prestressed wires. Hoyer was interested in acquiring the license of my process and visited me several times during the years 1927-1931. He got acquainted with all pertinent details in my factory in Most, especially gages and quality of piano wires, intensity of prestressing, vibration methods, curing, hardening, cutting, etc. Loading tests were made in his presence, and all details of manufacturing were revealed to him. Under Nazi Germany his manufacturing plant was largely subsidized by the government, with contributions of over 4 million marks. He never referred to my process and patents, and finally, with the support of the Nazi party he was granted his patent. Freyssinet's and my own protest were rejected in the Fascists' country. Freyssinet is, however, using wire rods instead, which is quite different. My prestressed concrete units manufactured and used in construction since 1919 are not only a pioneer work in this field, but they still are products of the highest perfection."

## APPENDIX†

Wettstein's prestressed concrete slabs (planks) are characterized by prestressed wire reinforcement arranged in both directions, longitudinal and transverse. The boards are either plain with a minimum thickness of approximately $\frac{3}{8}$ in. (10 mm) or with an insulating core. High-strength wires from 20 to 25 gage (1.0—0.5 mm) are used, with top and bottom reinforcement in the boards with insulating interlayer. It has been found by tests that the strength of such wire increases as the thickness (diameter) decreases. The wire reinforcement is prestressed while embedded in the surface layers of the boards and the pre-tensioning is maintained during the setting and hardening period of the high-strength cement mortar. By this process compression in the concrete is produced with maximum values in the extreme fibers where tensile stresses due to bending occur. Cement mortar and concrete have low resistance against tension which is practically eliminated by prestressed wire reinforcement, and consequently no cracks occur in tension zones. In boards of greater thicknesses, an interlayer of lightweight concrete is used which not only reduces weight of the boards but also provides both thermal and accoustical insulation. The mats of reinforcing wires protrude about 1 in. around the edges of the boards thus making interlacing along the joints and anchorage of additional reinforcing bars within these joints possible. Perfect continuity is obtained by grouting the joints with cement mortar. Thin reinforcing wires have higher specific strength and due to their small thickness the lever arm of interior forces is considerably increased, which results in greater efficiency and bearing capacity of the board.

Two types of Wettstein's prestressed concrete boards are manufactured, Type S and Type D with characteristics as shown in Tables *A*, *B* and *C*.

Nails can be driven into Wettstein's prestressed boards. The typical size of the board is 19 x 77 in. (48 x 198 cm) which covers, including about 1 in. continuity joints to be grouted

*Abeles, P. W., *The Principles and Practice of Prestressed Concrete*, New York, 1950, Frederick Ungar Publishing Co.

†Translation of "Wettstein's Prestressed Concrete Planks," by J. J. Polivka, *Teyssler-Kotyska* (Technical Encyclopedia), V. 15, Prague, 1933, pp. 335-336.

**TABLE A—SLAB TYPE S WITH LOWER REINFORCEMENT**

| Live load, lb per sq ft | Thickness of boards, in. | | | | | |
|---|---|---|---|---|---|---|
| | Span, ft | | | | | |
| | 3.3 | 4.0 | 4.6 | 5.2 | 6.0 | 6.5 |
| 15 | 0.8 | 1.0 | 1.2 | 1.6 | 2.0 | 2.4 |
| 30 | 1.0 | 1.2 | 1.6 | 2.0 | 2.4 | |
| 50 | 1.6 | 2.0 | 2.4 | | | |
| 80 | 2.0 | 2.4 | | | | |

**TABLE B—SLAB TYPE D WITH HIGHER REINFORCEMENT**

| Live load, lb per sq ft | Thickness of boards, in. | | | | | |
|---|---|---|---|---|---|---|
| | Span, ft | | | | | |
| | 3.5 | 4.6 | 6.0 | 7.0 | 8.5 | 10.0 |
| 15 | 0.4 | 0.8 | 1.0 | 1.2 | 1.6 | 2.4 |
| 30 | 0.8 | 1.0 | 1.2 | 1.6 | 2.0 | 2.4 |
| 50 | 1.0 | 1.6 | 2.4 | | | |
| 80 | 1.6 | 2.4 | | | | |

**TABLE C—WEIGHT OF SLABS**

| Thickness, in. | 0.4 | 0.8 | 1.2 | 1.6 | 2.0 | 2.4 |
|---|---|---|---|---|---|---|
| Type S, lb per sq ft | 3 | 4.3 | 6 | 7 | 8.5 | 9.8 |
| Type D, lb per sq ft | 3.3 | 4.7 | 6.5 | 7.6 | 8.9 | 10 |

after setting, 10.8 sq ft (one sq m). Boards of other dimensions are either specially manufactured, or cut out in the plant or on the construction site. The boards for roofs and floors are attached to the rafters and joists with special steel dowels anchored at the supported edges. Permanent anchorage of these dowels is obtained by successive grouting of the joints. Expansion joints are provided for lengths exceeding 80-100 ft to give weathertight, puttyless construction. No special roofing is required for slopes of more than 8 in 12 where waterproofing wash or fluate provide sufficient protection against rain. The Wettstein's prestressed and insulated boards are used also for walls and partitions (partitions in homes 2.4 in. thick).

## AUTHOR'S CLOSURE

Polivka's discussion is interesting as it contains valuable information concerning the contributions of Karl Wettstein in the development of prestressed concrete, hitherto undisclosed in any European or American publications which have come to the author's attention.

During recent discussions in New York with Paul W. Abeles, London, he corroborated the information given concerning the work of Wettstein which antedates the earliest publications of Freyssinet and Hoyer.

The author also takes this opportunity to mention a further omission in the paper concerning the notable contributions in the development of prestressed concrete made by Abeles, particularly in the field of "partial prestressing." The theory of partial prestressing has recently been used by Abeles with marked success and economy in the construction of a number of bridges for the Eastern Region of British Railways, and several articles have recently been published in *Civil Engineering and Public Works Review*, England, on the work.

Precast concrete techniques reduced building costs for owner and contractor.

# Precast Concrete Panel Multistory Construction*

By THOMAS F. GILBANE†


## SYNOPSIS

The construction procedure by which precast concrete panels were used in multistory buildings is described. Brief information is given on other structures where similar construction methods were utilized. Because of inherent economies, the author predicts wider use of precast concrete construction.


## INTRODUCTION

The use of precast concrete panel type construction in multistory structures to many is unbelievable. However, to those of us who have used this type of construction to build multistory buildings, it is indeed a reality, and in the author's opinion, much more will be heard of this type of construction in the years to come.

In practically all engineering magazines there are articles on precast panel type multistory construction, tilt-up precast concrete, and precast wall sections. All of these methods are being used, and descriptions of the time and money saved in their utilization are of great interest.

The possibilities of increased production and lower costs in construction through standardization has heightened interest in precast concrete construction. As many know, precast concrete units have been successfully used in Europe for some time, and are finding increasing application in this country.

The author's firm has been privileged to have been associated with several of the country's leading architects and engineers in the construction of multistory precast concrete projects such as dormitories at the University of Connecticut, designed by McKim, Mead and White, architects, New York, and Fred N. Severud of New York, structural engineer; and the City of Manchester Housing Project,

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†President, Gilbane Building Co., Providence, R. I.

Provost and Wright, Inc., architects, Manchester, N. H., and Frank Whelan of Boston, structural engineer. The architects and the engineers are to be congratulated for their farsightedness in the preparation of the plans and specifications for these units. The structural engineers did an extraordinary job in designing the structure as they kept to the minimum the reinforcing steel, and simplified the job for the contractor by the economical arrangement of the structure.

## MULTISTORY DORMITORIES

Dormitory units at the University of Connecticut are for the most part four-story and basement structures built entirely of load-bearing precast wall sections (Fig. 1). No structural steel was used. On the original dormitory units no reinforced concrete columns were required. For other units, reinforced concrete columns were used for the first floor lounge areas as well as in the dining rooms to provide for larger recreational space.

The major reinforced concrete beams in the project are the spandrel beams required at each floor level. All of the walls, bearing and non-bearing, are precast concrete panels. Exterior and corridor walls comprise the bearing structure upon which reinforced concrete floors were cast in place. Concrete foundations for the structures were cast in place below grade. The basement walls above grade are precast 5-in. sand cast panels backed-up by 8-in. precast panels which carry the load of the superstructure and present a finished interior concrete surface. A 2-in. air space separates these two exterior panels. Exterior wall panels are 8 in. thick except under the windows where 4-in. panels are utilized. Corridor partitions are 6 in. thick above the basement; the partitions between the bedrooms are 4 in. thick. Exterior of the building is brick veneered. Limestone slip sills are used under window frames, and

**Fig. 1—Dormitories at the University of Connecticut. In the connecting unit in the background the precast concrete panel construction can be seen. The units in the foreground are of similar construction on which brick veneer has already been applied.**

brick veneering is attached to precast exterior wall panels by galvanized anchors tied to dovetail galvanized inserts placed in the exterior surface of the outside wall panels.

The engineers and architects contributed greatly to the economy of operation, by utilizing standard lengths and heights of panels. Typical exterior wall panels were 8 ft long and story height less the depth of the spandrel beams. The interior partitions averaged 14 ft in length, and were a full story high. Naturally some of these dimensions varied in particular instances.

In all cases, the precast panels extend up into the cast-in-place floor slab or the spandrel beam, as the case may be, a minimum of ½-in. anchorage (Fig. 2). A 2-in. air space separates the exterior brick veneering, thus insulating the buildings. Weep holes are provided in the exterior masonry.

Roof slabs are covered with the usual insulating materials, and for 18 of the 20 dormitories, tar and gravel roofing was applied to the flat roof surfaces. Two of the dormitories have pitched roofs composed of a lightweight structural steel framework supporting gypsum plank and slate roofing.

All interior surfaces of precast panel walls were painted directly upon the concrete with the exception of stair halls and toilet rooms where glazed tiles were attached to the precast panels by dovetail inserts and galvanized anchors.

All door bucks were of steel, dropped in place before the floor above was placed. The door bucks were grouted in place, while all interior joints were caulked with dry oakum and pointed. The assembly of the panels makes these joints comparatively inconspicuous.

The precast corridor panels are continuous from door buck to door buck; therefore, the only joints in the corridor occur in the lintel stone over the door frame while in the rooms the joints occur between the connection of exterior wall panels and separating partitions. This is also true where the separating partition meets the corridor partition.

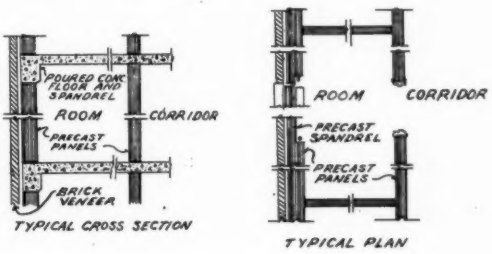


**Fig. 2—Method of incorporating precast panels in multistory construction**

Floors are generally covered with asphalt tile, while ceilings are either of rubbed concrete or acoustical tile applied directly to the underside of the floor slab.

As in any construction project, the scheduling and planning of operations is of great importance. Complete and detailed shop drawings are necessary prior to the start of any precasting operations. It is absolutely essential to properly locate all electrical conduit, pipe chases, drinking fountains, fire extinguisher cabinet openings and any other specialty items that require proper and definite location. The rooms must be properly scheduled and job requirements analyzed clearly so that when precasting operations start a continuous flow of panels keeps production costs at a minimum. Setting plans are necessary and are a great aid to the erection crews.

The contractor has used steel edge forms, sometimes supplemented by wood, and casting beds, preferably of concrete, in casting operations. The stack or sandwich method of precasting is employed. Under this method, edge forms are erected on the casting beds and a liquid separator is applied. Then the necessary reinforcing steel, electric conduit and any other blocking or inserts are placed before the concrete is placed, the top surface given a smooth trowel finish, and allowed to cure until the following day. The forms are then removed and may be immediately reversed. Canvas separators have also been used to good advantage.

It is of great importance to properly cure the panels by controlled atmospheric conditions. This can best be accomplished by the construction of a casting plant such as that erected at Manchester, N. H., where curing conditions were properly controlled.

Lifting hooks were inserted in the edges of the precast panels to allow for lifting from the casting beds and subsequent erection in the buildings. The precast panels were placed with crawler and truck cranes. The contractor accurately laid out lines of corridor partitions and separating partitions on the cast-in-place floor slab, then the floor centering was erected, leaving slots for insertion of precast panels in proper places. All precast panels were set in a bed of mortar to allow for even distribution of the weight of the slabs.

The structure is tied together by the steel handling hooks, set in the top of the panels when cast, and the top half inch of the precast panels extending into the cast-in-place floor above.

Exterior panels extend into the spandrel beams between each floor. The spandrel beams are placed monolithically with the floor slabs. Flat floor slabs are continuous and span from exterior to corridor walls. The floor serves as a platform on which the next precast slabs are erected. 2500 psi concrete is utilized throughout the floor slabs and precast

panels. Upon completion of the erection of the precast panels and the cast-in-place floors, finishing operations can immediately start.

Hanging scaffolding is utilized to place exterior masonry while mechanics are completing inside finishing operations such as hanging doors, applying asphalt tile and acoustical tile ceilings, painting and general clean-up. In a comparatively short time the building is ready for occupancy.

Several thought that the acoustical qualities of this type of construction would be inadequate. However, it has been found that the bedrooms with the usual furnishings and draperies created satisfactory acoustical conditions. All corridors in the dormitories have acoustical tile ceilings.

Precast concrete multistory construction has proved to be permanent, fireproof and of exceptional low cost. The possibilities of increased production and lower costs through speed of construction have interested many in precast panel type construction.

## YOUTZ-SLICK METHOD

Another interesting development which utilizes many of the features of precasting, is the Youtz-Slick method of precasting several floors and the roof structure of the building in stack formation, and then lifting the roof and the respective floors horizontally to their proper location by use of tension rods that straddle H columns that have been erected prior to casting floors and the roof. Yokes are placed on the top of the column and with hydraulic jacks the roof slabs and floor slabs are raised to their proper level and attached to the steel column by a steel collar placed around the H column when the floors and roof were precast. It is understood that four houses utilizing this method have been constructed at Yorktown, and the largest floor slab lifted to date was 38 x 44 ft.

At the present time, Mr. Severud is designing a two-story building utilizing this method to be constructed on the West Coast. Under this method, not only the wall panels, separating partitions or corridor panels are to be precast, but the entire floors and roof of this structure will be precast with the possible exception of certain connecting sections of floor and roof slabs that for ease of operation will have a six-foot opening left the length of the building. These openings will be cast in place in sliding forms, thus insuring proper tie of all roof and floor sections of the building. The precast panels are to be placed with fork-lift trucks.

## ECONOMIES OF PRECAST CONSTRUCTION

It can be readily seen that there are many economies in the use of precast concrete multistory construction, for in utilizing edge forms

instead of the usual method of vertical forming, savings in materials and labor can be realized. The substitution of precast panels for cast-in-place concrete or built-up masonry and the use of other materials requiring field erection of individual units eliminates many man hours of skilled labor. The precasting of panels under cover in a casting plant eliminates to a certain extent the contractor's greatest handicap —weather; a good step taken in the right direction.

Technical control is better in the casting plant than under normal job-site conditions, and the speed of erection can in itself be a determining factor for the use of this type of construction. With proper control, the influence of shrinkage can be practically eliminated. The use of expansion joints normally required in cast-in-place concrete can be eliminated.

Several interested people have inquired as to the limits in height of precast concrete buildings. They have believed that this type of construction is limited to three or four stories. At the University of Connecticut, the central unit is six stories with a full basement and large dining room directly below the living quarters. Fred N. Severud, one of the outstanding concrete designers in the country, has confirmed the author's opinion that precast panel type construction need not be limited to four, five or six stories in height.

Naturally, the placing of the precast panels by crane is limited by its boom length and radius. However, with the aid of construction equipment such as Chicago booms and derricks, the lifting problem is easily solved, and with the increased use of lightweight aggregates it is safe to predict that the use of precast panel construction will greatly increase in the future.

Many have inquired as to the potential savings of this method of construction. From actual figures comparing first-class fireproof construction with multistoried precast type construction, savings up to 30 percent are indicated. The scarcity of skilled mechanics, such as bricklayers and plasterers, in many parts of the country has stimulated the use of this type of construction. Actually, the higher wage scale for bricklayers and plasterers is a factor in determining the comparative cost. However, the speed of operation is of major importance, as the building can be occupied much sooner than when usual construction methods are used. In institutional construction, one of the major considerations is maintenance cost. Through the use of precast concrete panels, sizable savings can be realized, as maintenance cost of this practically indestructible material is low.

It is interesting to note that a crack-free concrete can be developed with proper handling in the early hours of casting. Recent experiments by Fred N. Severud indicate that placing precast panels under horizontal

compression in the early hours of curing, results in crack-free concrete which can be utilized to great advantage for exterior as well as interior use.

The prospective user of multistory precast concrete panel construction, although utilizing one of the most modern methods of construction, can through surface treatment of the exterior walls and roof create a beautiful Georgian Colonial, Gothic or modernistic type of architecture.

Precast concrete construction can be readily adapted to hospital, apartment, hotel and all other types of construction. Naturally, the savings in time and money will be greater where repetitive cubical type construction is utilized. The continuous research by many engineers, architects and contractors on the use of lightweight aggregates or foam concrete, such as now used in Germany, is a guarantee that the technique of precasting and erection of multistory precast concrete structures will continue in the future.

Further data on tests of precast concrete floor joists initiated by ACI Committee 711

# Tests of Precast Reinforced Concrete Joists*

By C. D. WILLIAMS† and F. BROMILOW‡


## SYNOPSIS

Tests made as part of the work of ACI Committee 711 to determine the effect of bar spacing on the strength of precast concrete joists are described. The method of quarter-point loading of commercial joists and the results of tests of bond specimens are briefly reviewed. It is concluded that the strength of the joists is controlled by the character of the weld used to fasten the stirrups to the main steel and that the test method used indicated no correlation between effective bond area of the steel and the load carrying capacity of the joists.


## INTRODUCTION

Early in 1949, ACI Committee 711 (F. N. Menefee, University of Michigan, Chairman), decided to have a series of precast concrete floor joists made and tested to discover the effect, if any, of the close spacing of moment bars. The joists, as manufactured, have reinforcing bars with a clear distance of $\frac{1}{8}$ to $\frac{1}{2}$ in., determined by the size of stirrups used. ACI Building Requirements for Reinforced Concrete, Par. 505(*a*), requires a minimum clear spacing of one inch.

The joists to be tested were to be made according to common manufacturing practice but were to have some of the bond area of the tensile steel removed by painting part of the surface of these bars with a non-bonding substance. Two manufacturers agreed to furnish joists for such a test program. One of these, Maule Industries, Inc., Miami Beach, Fla., was so far from Michigan that the University of Florida was asked to undertake the tests.

From the beginning it was feared that the desired results might not be obtained from the tests on these joists. The possibility that failure of the joists might occur due to shear or flexure was considered. Con-

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sideration was also given to the possibility that all of the joists might fail in bond, in which case the effect of reduction of effective bond area would be obscured. It was decided that, in addition to the tests on the precast joists, an independent study of the effect of spacing of bars would be initiated. This study was to be pull-out tests of bars with a range of spacings in concrete which matched as nearly as possible that in the joists. Two parallel studies were made, one using the precast joists, and the second using pullout specimens prepared at the university. These tests are described separately, but both are considered in making the final recommendations.

## JOISTS AND TESTING EQUIPMENT

### Joists

The reinforcing steel used in the joists tested was of structural grade with an assumed yield point of 33,000 psi. The concrete, as described by the Maule Co., was "1 cement: 2 rock screenings: 2 ¼-in. down. Batching was by volume, a 2-bag mix being used."

Crushing strengths as measured by the Pittsburgh Testing Laboratories were:

| Age, days | Crushing strength, psi | Slump, in. |
|---|---|---|
| 4 | 2625 | 2½ |
| 7 | 3004 | |
| 28 | 4865 | |

At the time of testing, data had not been received on the 28-day strength. For preliminary analysis the 28-day strength was estimated as 4500 psi.

All the joists tested were 3½ in. x 8 in. x 10 ft having two ⅜-in. round rods in the top with 1-in. clear cover and two ¾-in. round rods in the bottom with ¾-in. clear cover.

Fourteen joists were tested and of these, 11 had vertical stirrups of ⅜-in. round rods and three had vertical stirrups of No. 6 wire. The stirrups were tack welded to the longitudinal steel, both top and bottom. The over-all length of the longitudinal reinforcing steel was 9 ft 10 in. Fig. 1 shows details of the steel assembly and of the cross section of the joists.

For the purpose of this research, varying percentages (0 to 30 percent) of the bond area were rendered inoperative in some of the joists. The bond resistance was destroyed during the make-up of the joists by coating the proper portions of the tensile steel with a resinous material which gave the coated portion a smooth nonbonding surface. Of the 11 joists with ⅜-in. round stirrups two had no bond destroyed, three had 10 percent bond destroyed on the tension rods, three had 20 percent bond destroyed on the tension rods and three had 30 percent bond de-

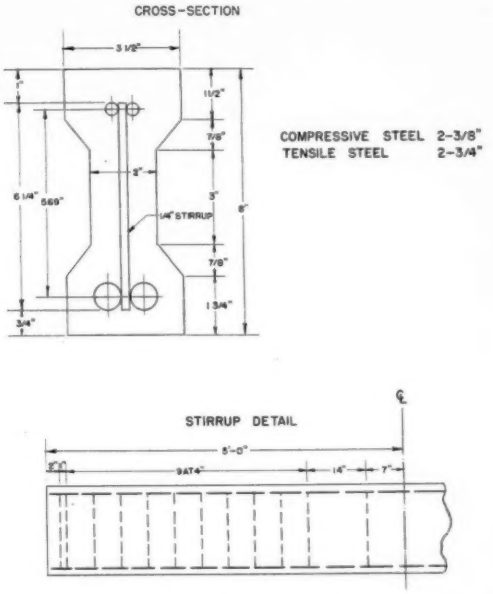



**Fig. 1—Details of joists tested**

stroyed on the tension rods. The three with No. 6 wire had 30 percent bond destroyed.

### Testing method

The ACI Building Code recommends the use of a test load uniformly distributed over the entire length of the joists—the joist being simply supported at the ends. The difficulty in controlling the rate of application of a uniform load made it necessary to select another type loading. It was decided to use equal concentrated loads at the quarter points of the joist. This gave the same maximum moment and maximum shear relationship as for a joist loaded uniformly throughout its length. Fig. 2 shows the shear and moment diagrams for the uniform and quarter-point loadings. The end shear value for the quarter-point loading is maintained from the end to the quarter-point of the joist. Therefore, the maximum bond stresses are maintained throughout this length. The maximum moment is equal for both loadings.

A 400,000-lb Riehle, screw-type testing machine was used in all tests. To apply loads at the quarter-points, an 8-in. I, 18.4 lb section was used as a load beam. The head of the testing machine applied load at the mid-point of this load beam which was supported on ½-in. round steel rods resting on ¼-in. steel bearing plates placed at the quarter-

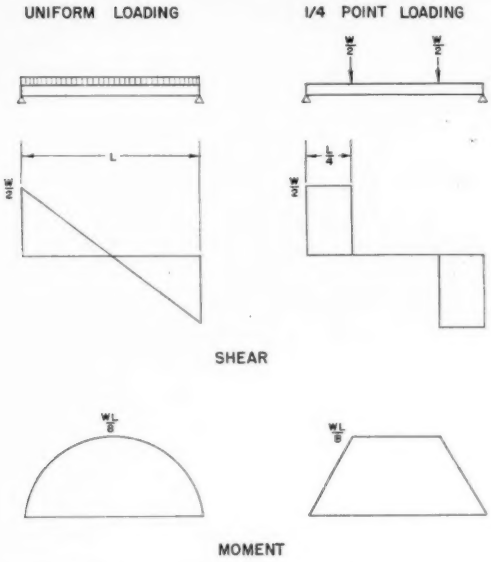



**Fig. 2—Shear and moment relationships for uniform and quarter-point loading**

point of the concrete joist. The concrete joist was supported at its ends on roller bearing nests 6 in. wide. The roller bearing nests rested on rigid knife edges set 3 in. from the ends of the concrete joist. The load was applied at the slowest possible no-load speed of 0.05 in. per minute. A $\frac{3}{8}$-in. cap consisting of a half-and-half mix of gypsum and portland cement was used between the ends of the joist and the roller bearing nests to insure proper bearing. Details of the loading arrangement are shown in Fig. 3.

The load was applied so that the end shear values increased in increments of 500 lb. The deflection of the center of the joist relative to the ends for each increment of loading was obtained by using Ames dials. One dial was placed at the center and one at each end. To eliminate creep caused by plastic flow in the concrete and compression of the capping material, each joist was loaded repeatedly from zero load to the design load and then back to zero load. This cycle was continued until the deflection as measured by the dial readings remained the same for each design load repetition. After this had been done, the load was increased from zero to failure load in 1000-lb increments with deflection and crack observations being made after each increment.

## CALCULATION OF PROPERTIES OF JOISTS

To arrange proper testing procedure it was necessary to compute the

**Fig. 3—Details of quarter-point loading arrangement**

neutral axis, moment of inertia, allowable bending moment, allowable shear stresses and bond stresses of the concrete joists.

**Assumed values**

Ultimate compressive strength, 4500 psi ($f'_c$)
Allowable compressive strength, 2025 psi ($0.45\,f'_c$)
$n = 7.0$
Area of compressive steel = 0.22 sq in.
Area of tensile steel = 0.88 sq in.
Yield point of steel = 33,000 psi
Allowable unit stress in the steel = 18,000 psi
Total depth of cross section = 8 in.
Clear cover of compressive steel = 1 in.
Clear cover of tensile steel = ¾ in.
Span length = 114 in.

**Calculated values**

$kd = 3.4$ in.
$I = 125.21$ in.$^4$ (For transformed section)

**Computation of allowable design values**

Allowable bending moments for concrete and steel are:

$$M_c = \frac{f_c I}{kd} = \frac{2025 \times 125.21}{3.4} = 74{,}700 \text{ in.-lb}$$

$$M_s = \frac{f_s I}{n(d\text{-}kd)} = \frac{18{,}000 \times 125.21}{7 \times 3.48} = 92{,}500 \text{ in.-lb}$$

Specifications require that the individual joist withstand, without complete failure, a uniformly distributed load of at least 2.25 times the design live load based on allowable stresses in bending and shear

as given by the code. The bending moments that the joist must sustain are $M_c = 74{,}700 \times 2.25 = 168{,}000$ in.-lb and $M_s = 92{,}500 \times 2.25 = 208{,}000$ in.-lb. If failure occurs from bending, the concrete would be expected to fail before the steel reinforcing. End shear values necessary to give these moments are computed by dividing the moment by $L/4$, which is 28.5 in., and are:

For the concrete, end shear $= \dfrac{168{,}000}{28.5} = 5900$ lb

For the reinforcing steel, end shear $= \dfrac{208{,}000}{28.5} = 7300$ lb

#### Shear computations

Unit shear may be computed from the standard formula, $v = \dfrac{VQ}{It}$

when $v$ = unit shear.

$V$ = total shear on the section.

$Q$ = static area moment of the section outside the plane where the shear is desired.

$I$ = moment of inertia of the section acting.

$t$ = thickness of the section at which $v$ is desired.

Below the gravity axis, the value of $Q = 6.16 \times 3.48 = 21.5$ The maximum unit shear due to a 1000 lb shear load is then

$$\frac{1000 \times 21.5}{125.21 \times 2} = 86 \text{ psi}$$

The Code allows a shear value of $0.06\ f'_c$ for beams with properly designed web reinforcement but without special anchorage of the longitudinal steel. The allowable design value of $v$ is then equal to $0.06 \times 4500 = 270$ psi. If the shear stress per 1000 lb of shear load is 86.0 psi and the design shear stress as given by the Code is 270 psi, then a ratio may be used to compute the corresponding shear load.

$$\frac{1000}{V} = \frac{86}{270}, \text{ then } V = \frac{1000 \times 270}{86} = 3140 \text{ lb}$$

The Code states that the test joists must withstand live loads 2.25 times the design live load. The shear is directly related to the loading, therefore, the shear load that the joists must sustain is equal to $2.25 \times 3140 = 7070$ lb.

#### Bond computations

The allowable unit bond stress for joists constructed with deformed bars is given as $0.05\ f'_c$ with a maximum of 200 psi. The total shear developed by 1000 lb of shear load is equal to 172 lb per in. at the plane of the tensile steel. It is reasoned that this shear must be transferred

by bond to the tensile steel. The bond force per 1000 lb of shear load is then equal to 172 lb. The tensile steel has 4.72 sq in. of bond area per lin. in. of beam, hence the bond stress for a shear of 1000 lb is

$$\frac{172}{4.72} = 36.5 \text{ psi.}$$

The shear load that will produce 200 psi bond stress is then $\frac{200}{36.5} \times 1000 = 5470$ lb. When the specification safety factor of 2.25 is applied to bond, the shear load may be $2.25 \times 5470 = 12{,}300$ lb.

#### Predicted failure

From these computations it can be seen that the section should be expected to fail in shear with a shear load of 7070 lb, in bond with a shear load of 12,300 lb, or in compression where the bending moment reaches 168,000 in.-lb.

The maximum end shear with quarter-point loading which would cause a moment of 168,000 in.-lb is 5900 lb as previously indicated and since this is the smallest of the three controlling values it would be expected that the beam would fail in flexure. However, it has long been known that a redistribution of stress takes place within a concrete beam before failure, and that because of this, the flexure formula as commonly applied is no longer valid. If it is assumed that the neutral axis does not shift before failure, there is still the matter of the variation of the stress in the concrete to be taken into account. This has been discussed* and a number of theories advanced with respect to the shape of the stress-distance from neutral axis curve. Assuming that in a rectangular beam the stress distribution at failure is a parabola with the vertex at the extreme fiber, when the stress is 4500 psi, the moment will be 216,000 in.-lb and the end shear 7580 lb. This is higher than the value at which shear failure is expected and therefore a shear failure in the beam can be predicted.

The stress in the steel at the design load is 18,000 psi. If the load is increased 2.25 times, the straight line flexure formula would give the result of 40,500 psi in the steel. Since the yield point is less than 40,500 this condition cannot exist and stress redistribution must occur before 2.25 times the design load has been applied.

## TEST RESULTS OF JOISTS LOADED AT QUARTER POINTS

Joists 1 through 9 were each tested by loading at the quarter points as previously described. After the design load had been applied and removed a number of times to remove any errors in starting the readings,

*Reinforced Concrete Design*, Sutherland and Reese, 2nd Edition, John Wiley and Sons.

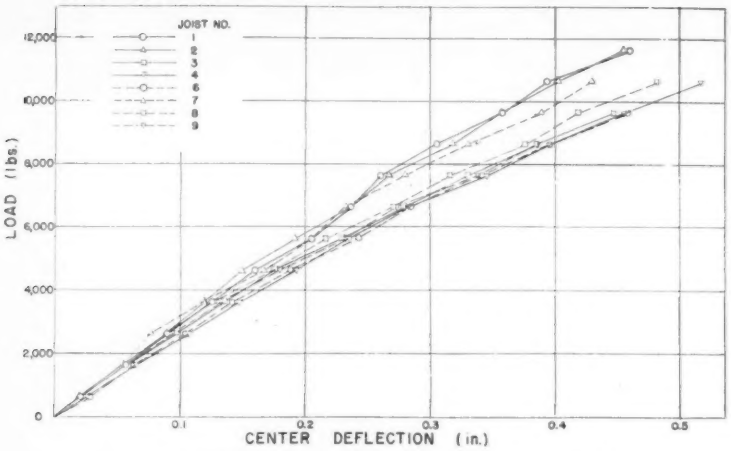



Fig. 4—Loads and deflections of joists loaded at quarter points

the load was increased in 1000-lb increments. At each increment in load, the deflection of the joist was measured and a close observation made of the joist to locate cracks in the concrete. The values of deflection and the location of cracks were recorded for each increment of load for each joist. Fig. 4 is a graph showing the relation between load and deflection. The description of the crack formation for each joist was kept for purposes of indentifying the type of failure. These descriptions have not been reproduced here except the final condition at failure for each joist. (Table 1).

Fig. 5 and 6 show eight of the first nine joists after testing. The cracks in the joists have been outlined in black to show their location.

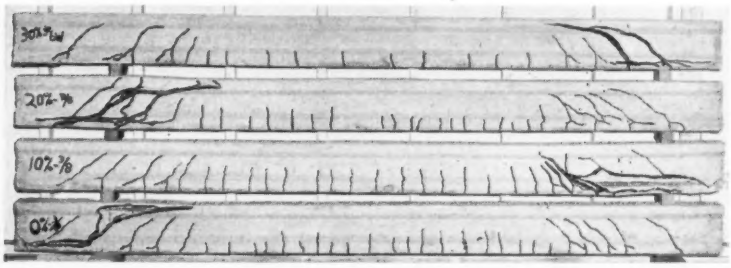


Fig. 5—Crack pattern in joists loaded to failure at quarter points

**TABLE 1—TYPE OF FAILURE IN JOISTS—QUARTER-POINT LOADING**

| Joist No. | Type of stirrups | Bond removal, percent | Load at failure, lb | Description of failure |
|---|---|---|---|---|
| 1 | ⅜-in. | 30 | 12,370 | Definite shear failure. |
| 2 | ⅜-in. | 20 | 13,930 | Definite shear failure. |
| 3 | No. 6 wire | 30 | 9622 | Existing shear cracks extended to the top flange and opened up still more. The shear crack at 18 in. from the end of the joist was about ¼-in. wide. While observing the cracks, a definite shear failure occurred. |
| 4 | ⅜-in. | 10 | 13,470 | Definite shear failure. |
| 5 | ⅜-in. | 0 | 13,400 | Failure by shear. Horizontal cracks occurred along the bottom flange, but only after the shear cracks were well developed. |
| 6 | ⅜-in. | 30 | 9622 | Existing shear cracks widened. Several more horizontal cracks appeared. Definite shear failure. |
| 7 | ⅜-in. | 20 | 12,100 | Definite shear failure. |
| 8 | ⅜-in. | 10 | 11,270 | Definite shear failure. The crack was at 45° through the web. A horizontal crack appeared in the lower flange at failure. |
| 9 | No. 6 wire | 30 | 10,420 | Definite shear failure. |

The first joist tested is not shown. It was broken apart immediately after testing to get accurate measurements on the reinforcing steel.

## TEST RESULTS OF JOISTS LOADED AT CENTER

After 9 of the 14 joists had been tested, each failing in shear, it was reasoned that no bond failures could be expected under any type of

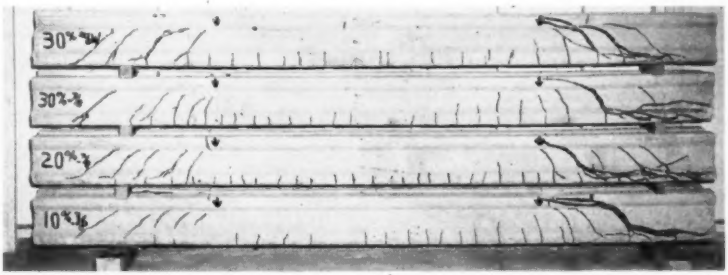



**Fig. 6—Crack pattern in joists loaded to failure at quarter points**

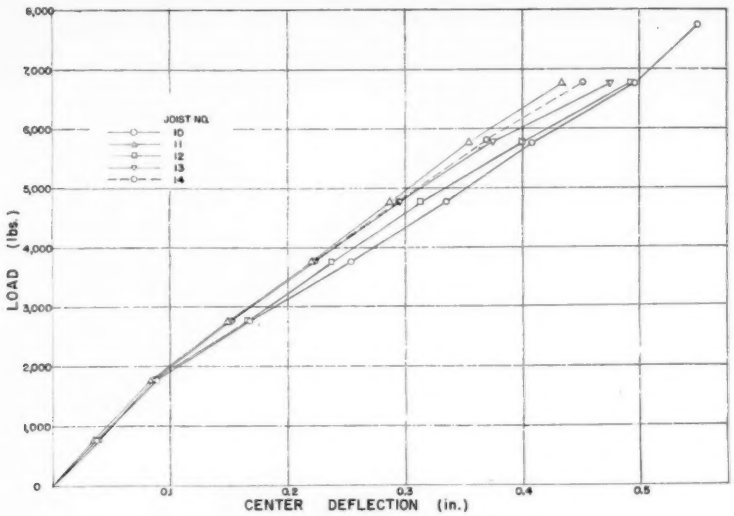



**Fig. 7—Loads and deflections of joists loaded at center**

loading. In an effort to get a different relationship between flexural stress and shear stress, the remaining 5 joists were loaded with a concentrated center load.

Test procedure and recording of data was the same as that used on the first nine joists. Fig. 7 is a graph showing the relation between load and deflection for each joist. The description of the failure of each is given in Table 2. Fig. 8 shows the joists after testing. The cracks are outlined in black to show their location.

**TABLE 2—TYPE OF FAILURE IN JOISTS—CENTER LOADING**

| Joist No. | Type of stirrups | Bond removal, percent | Load at failure, lb | Description of failure |
|---|---|---|---|---|
| 10 | ⅜-in. | 10 | 7500 | Failed in shear near center, |
| 11 | ⅜-in. | 20 | 7500 | Failed in shear. |
| 12 | ⅜-in. | 30 | 7960 | Shear failure. Immediately afterwards, the concrete fell off below the tensile steel. |
| 13 | No. 6 wire | 30 | 7450 | Definite shear failure between the center and the quarter-point. Indication of bond failure but after joist had failed from shear. |
| 14 | ⅜-in. | 0 | 8130 | Failed in shear. |

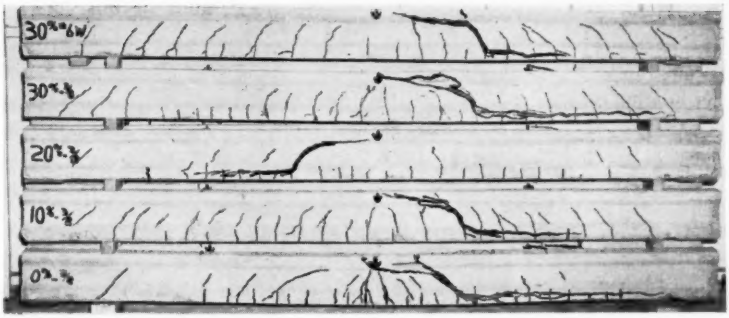


Fig. 8—Crack pattern in joists loaded to failure at center

## DISCUSSION OF TEST RESULTS

Since all of the tested joists failed in shear, no data are available from this study on the bond strengths of the joists. A study of the computed strength reveals the reasons for this shear failure. The maximum end shear that the joist could be expected to carry was 7070 lb, based on sheer stress of 0.06 $f'_c$. Of this total value it is expected that the concrete in the web (based on 0.02 $f'_c$ stress) will carry 2357 lb of this end shear. The remaining 4713 lb must be carried by the stirrups. An examination of the anchorage of the ends of the stirrups to the longitudinal steel disclosed small welds which, after failure of the joist, had pulled apart. If, for purposes of analysis the weld was neglected, the strength of the stirrups is limited by the bond that they can develop above the mid-height of the joist. Using an allowable bond stress in the $\frac{3}{8}$-in. deformed bar stirrups of 0.05 $f'_c$ and an allowable bond stress of 0.04 $f'_c$ in the No. 6 wire stirrups with a spacing of 4 in. for both types a calculation of the capacities of these stirrups was made. The factor of 2.25 was used to convert these allowable values to predicted failure values in accordance with the methods used earlier. The additional reaction that could be carried in shear by the stirrups under these conditions was then computed. This value is 2600 lb of end shear for $\frac{3}{8}$-in. deformed bars and 1065 lb for plain No. 6 wire stirrups. For the joists with $\frac{3}{8}$-in. stirrups this gives expected end shears at failure of (a) with stirrups not welded—4957 lb and (b) with stirrups with good welds—7070 lb. It is noted that all joists failed in shear within this range, the lowest at 5000 lb and the highest at 6965 lb indicating that full strength in the stirrups was not developed in every case. In the joists with No. 6 wire stirrups the expected end shear at failure was (a) with stirrups not

welded—3422 lb and (b) with stirrups with good welds—7070 lb. Again the joists tested failed within these limits, one failing at an end shear of 5000 lb and the other at 5200 lb.

The above discussion deals with joists loaded at the quarter points. The joists subjected to a concentrated mid-point loading also failed in shear. In each case the failure occurred in the central portion of the joist where stirrups were spaced at 14 in. Since these stirrups are too far apart to be considered effective according to ACI specifications, the strength of the joist was limited by the shear in the concrete which gave a predicted end shear at failure of 2367 lb. If the stirrups are considered to assist in the resistance to shear, the $\frac{3}{8}$-in. stirrups would allow the shear to go to 7070 lb when the weld is assumed to be good and to 3117 lb if strength is based on the bond value of the stirrups themselves. Failure in these joists occurred between 3750 and 4095 lb of end shear. Similar calculations using No. 6 wire stirrups result in predicted shear failure of 2672 lb when bond on stirrups controls and 7070 lb when the stirrups are considered to be fully effective. The one joist tested failed at 3725 lb.

In all cases no correlation was noted between the reduction of bond area and strength of joist. This is to be expected since the calculated end shear at which failure due to bond is expected is 12,320 lb, which is reduced to 8624 lb when the 30 percent reduction in bond area is

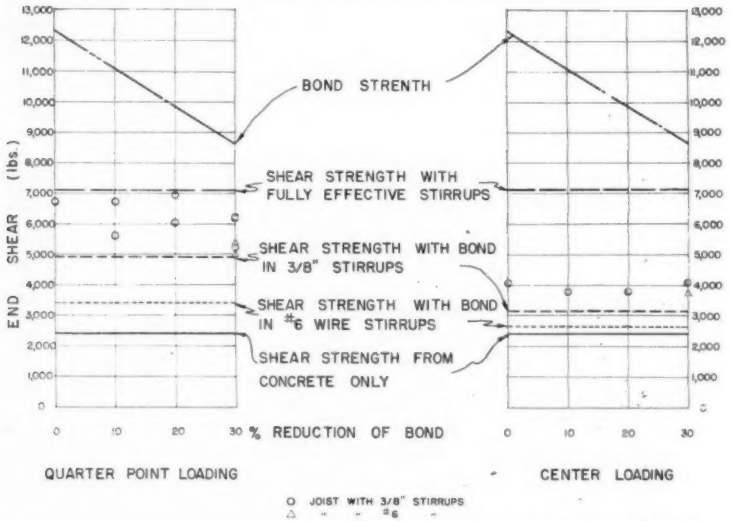



**Fig. 9—Comparison of bond and shear strength for quarter-point and center loading**

considered. Both of these values are far above the strength of the joist in shear. The relationships expressed in the above discussion are shown graphically in Fig. 9.

## PULL-OUT TESTS

The pull-out test was decided upon since it gives the best opportunity of studying bond properties unmasked by other variables. Two types of specimens were prepared; first, specimens with a single ½-in. rod in the center of a 4 x 4 x 12-in. specimen, and, second, specimens with two ½-in. rods in a single specimen. The spacings between these two rods were ¼ in., ⅜ in., ¾ in., 1 in. and 1¼ in. The rod used in every case was ½-in. deformed bar.

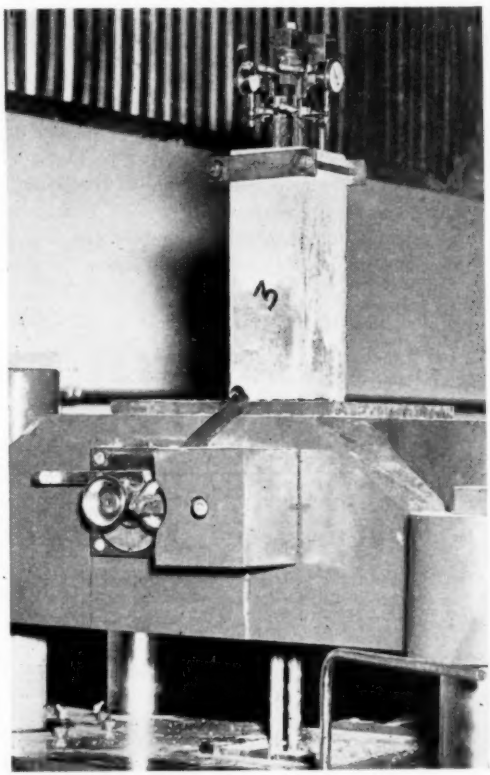


Fig. 10—Equipment arrangement for pull-out tests

The tests were made on a 300,000-lb Riehle hydraulic-ram testing machine. The slip of the bars was measured by a Riehle 2-in. averaging extensometer capable of direct reading to 0.0002 in. The machine used and the method of testing are shown in Fig. 10.

For preliminary work, a series of specimens were made with the two bars in each, spaced as mentioned above. These were tested at 7 days. The grip on the bars was made using a spacer block between bars and then inserting the pair in the standard wedge grip. Results of these tests (which are not included here) showed that one bar slipped before the other. It was felt that this would not give satisfactory results so, for the second series, the procedure was modified.

For the second series, two changes were made. First, the ends of the bars were welded to the spacer block before testing so that both bars would slip as a unit. Second, the series of spacings was reduced to ½, ¾ and 1 in. This was done to reduce the time needed to make specimens and to give approximately the same edge distance around the bars.

The specimens were cured for 28 days at which time the compressive strength of the concrete was 3920 psi. The bars acted as a unit, but one other difficulty became apparent, that of lack of good bearing between the concrete and the bed of the machine. These results, while not conclusive, are shown in Fig. 11. It will be noticed that the curves for the single bar are approximately the same as for double bars and that there is no significant difference between the bars of different spacings.

While these tests would lead one to believe that no bond strength was lost due to the close spacing of the bars, more tests along this line are needed. The technique used is believed to be satisfactory with the exception of the bearing between the concrete and the base plate.

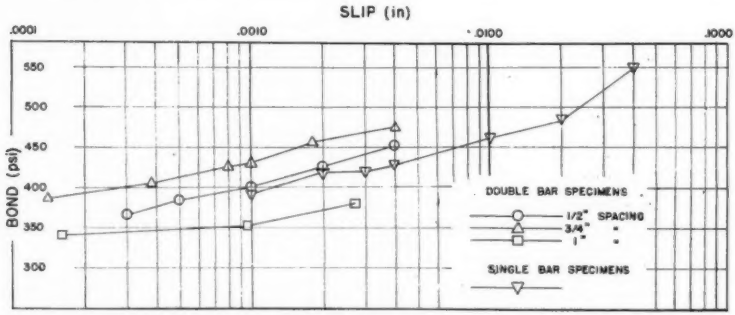



**Fig. 11—Results of pull-out tests on 28-day concrete**

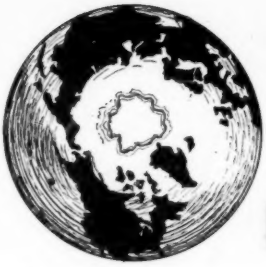
This could be corrected in subsequent tests by more careful methods in making the specimens.

## CONCLUSION AND RECOMMENDATIONS

1. These tests showed no effect of bar spacing nor did they eliminate the possibility of such effect on the strength of the joists. No correlation was noted between the reduction of effective bond area and the load carrying capacities of the joists. From these results, no recommendation is made concerning the spacing of tensile steel in precast floor joists.

2. The tests of the joists indicated that their strength is controlled by the amount and quality of the weld used to fasten the stirrups to the main steel.

3. The technique developed in the pull-out tests gives promise of providing the desired information on the effect of spacing of the bars. Further tests of this type should be undertaken.



# Letters from Readers*

## BY WAY OF SYNOPSIS

R. W. SPENCER compares dry mortar grout with expanding metallic grout.

CHARLES MACKLIN wants more data on form pressures.

The compression test is criticized by ANTHONY H. CLARK and JOHN H. THORNTON.

Crack control in portland cement plaster panels is discussed by L. E. JOHNSON.

G. L. BLANCHARD submits additional proof of the practicality of hexagonal bins.

Leaching of lime in a concrete arch dam is described by C. T. WOLLEY.

## Dry Mortar Grout vs. Expanding Grout (LR 46-55)

R. W. SPENCER, Southern California Edison Co., Los Angeles, Calif., cites experience with various types of grout.

An article in the JOURNAL on dry mortar grouting† and the ensuing discussion‡ brings to mind some experience with expanding metallic grout. A grout containing metallic aggregate has been used occasionally in machine foundations by the Edison Co. over the last 25 years with generally satisfactory results. However, a few years ago, such a grout was used to support the foundation base plates for a large outdoor synchronous condenser. The foundation was 74 ft long and the average depth of grout was $\frac{7}{8}$ in. Two years later the condenser was moved to a different location. At that time a check on the base plate showed warpages up to 0.014 in. due to irregular expansion of grout and steel shims under the plate. Had the machine remained in place a few years

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 9, May 1950, *Proceedings*, V. 46. Separate prints are available at 35 cents each. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Brooks, Boyd S., "Dry Mortar as a Bearing and Grouting Material," ACI JOURNAL, Jan. 1949, *Proc.* V. 45, p. 369.

‡Kaufman, R. R., Discussion of "Dry Mortar as a Bearing and Grouting Material," ACI JOURNAL, Part 2, Dec. 1949, *Proc.* V. 45, p. 380-1.

longer the expansion would have made necessary a very expensive realignment.

Apparently moisture entered the grout under the base plates, dissolved some of the salts in the grout, and caused excessive expansion of the grout itself as well as corrosion of the shims and base plate. From this experience I would judge that the use of a grout containing expanding metallic aggregate is likely to cause trouble at locations where moisture is present. In general, I believe the dry pack grouting method is an excellent one.

## Concrete Form Pressures (LR 46-56)

CHARLES MACKLIN, Structural Engineer, Springfield, Ill., wants more data on form pressures.

I would like to see published some authoritative and up-to-date information on concrete pressure in forms. Concrete pressure is difficult to determine and accounts for the conflicting ideas in reference books and in field practice. I have a collection of concrete pressure data and editorial comment dating back to 1894 and some of the differences of opinion almost led to name calling.

To illustrate what the man in the field is up against, in 1933 I designed some forms for a pressure equal to 150 $R$. The engineering firm wanted 150 $H$ which amounted to 1500 as against 450 psf. Data gathered during three months' research and study of tests failed to convince the firm of the correctness of my stand and an appeal was made to two independent sources for substantiation. One failed to answer and the other referred me to old tests of questionable value. As a consequence the much higher values were used, leading to the bankruptcy of the firm for which I worked. The principal cause was the overdesign of forms which cost $10,000 more than they should have.

A copyrighted formula is now in use which under the conditions of the work cited above also gives 450 psf with a constant varying from 90 to 150 inversely with the temperature. The point is this, had a usable formula been generally accepted and used in 1933 needless loss would have been avoided.

And think of the wasted money and materials used up since then in overdesigned forms. Isn't it about time that some reasonable design data, adequately backed up by experimental work, be made available to engineers so that they can design economical forms with some assurance that their computations can be authoritatively supported?

## Testing of Concrete (LR 46-57)

ANTHONY H. CLARK, Beckenham, Kent, and JOHN H. THORNTON, Hatfield, Herts., England, question the use of the compression test in checking concrete strength.

The prevalent method of checking the strength of concrete, by means of compression tests on specimens made from concrete out of the mixer, seems to be unsatisfactory in many respects.

The British Standards Institution lays down a procedure for the making and curing of specimens, but results merely give a comparison between different concretes made and cured in a given set of circumstances (including curing in damp sand at between 50 and 70 F) and take into account neither the degree of effectiveness of placing of the job concrete nor the actual site curing conditions.

We are aware that experiments are being conducted at the present time into possibilities of assessing the compressive strength of concrete by means of ultrasonic waves and also by reason of its physical hardness but we believe that a good deal of work still remains to be done before either of these proposed methods can be put into practice.

In the meantime, it seems to us that the value of the information obtained from compression tests would be higher if such tests were made on cores taken from the actual work and we believe this method is practiced in America. It is practiced to a small extent in this country but, only, so far as we know, in connection with concrete roads.

We should welcome any information any of your members are able to give us regarding the taking and testing of cores from concrete structures, with special reference to mass concrete dams.

## Crack Control in Portland Cement Plaster Panels (LR 46-58)

L. E. JOHNSON, Finishing Lime Assn. of Ohio, Toledo, Ohio, suggests a method of solving the problem of volume change in cement plaster.

In the October 1947 JOURNAL there is an article entitled "Crack Control in Portland Cement Plaster Panels,"* which is of such interest to us that we desire to comment upon it, even at this late date.

In the introduction of the paper, the statement is made that water spray, common at hydroelectric power plants, and condensation which occurs in dam galleries and gate chambers are detrimental to the relatively soluble lime and gypsum plasters.

Lime plaster is not soluble. If it was, it could not be used for brick mortars, lime stucco, or lime plaster where the humidity is relatively

*Hall, Bert A., "Crack Control in Portland Cement Plaster Panels," ACI JOURNAL, Oct. 1947, *Proc.* V. 44, p. 129.

high. On the other hand, gypsum plasters, in the presence of moisture, disintegrate and, for that reason, they are not used for stucco or for mortar exposed to atmospheric conditions. Cracking of portland cement plaster or portland cement mortar is due to the volumetric changes encountered with subsequent wetting and drying after the material is in place. This characteristic of cement mortar and cement plaster has been demonstrated in the past through leaky walls and through cracks appearing in stucco.

It is axiomatic that the volumetric changes in a cement mortar vary inversely with the lime content, and it is for this reason that the mortars approved by the A.S.A. provide different types, starting with a mortar containing 25 percent of lime up to a straight lime. The small amount of shrinkage developed in a so-called 1:1:6 mortar or a 2:1:9 mortar is a recognized characteristic and is widely used. For the same reason, cement stuccos and plasters, to overcome the volume changes, contain varying quantities of lime, due to local conditions and local customs.

The restraining of the cement plaster panels, as outlined in the paper, may reduce the shrinkage so long as the restraining agencies are greater than the stresses due to volume changes that are set up in the plaster. While restraining the panels may reduce the tendency of cracking, the volumetric changes with subsequent wetting and drying will not prevent the plaster from cracking over a long period and I believe an investigation will show this to be true. It seems to be pretty well authenticated that if a cement plaster is used, and it contains a predetermined amount of plastering lime, the problem of volume changes will be satisfactorily solved.

## Construction of Hexagonal Bins (LR 46-28)*

G. L. Blanchard, Consulting Engineer, Chicago, Ill., supplements previous letters on hexagonal bins.

In the January *Letters from Readers*, hexagonal grain elevators built in Africa were mentioned. Fig. 1 shows an elevator built in 1932 near the Fez (French Morocco) railroad yards. It was part of a vast program in North Africa, 1932-35, sponsored by the French authorities to provide wheat farmers with adequate storage facilities.

The structure is composed of 19 cells concentrically disposed around a central shaft of the same hexagonal section as the cells. The inside face of each side of the hexagon is 8 ft wide; the depth of the cells is 120 ft. The central shaft contains the elevating equipment and an ele-

*See also ACI Journal, Oct. 1949, *Proc.* V. 46, p. 141 and Jan. 1950, *Proc.* V. 46, p. 396.

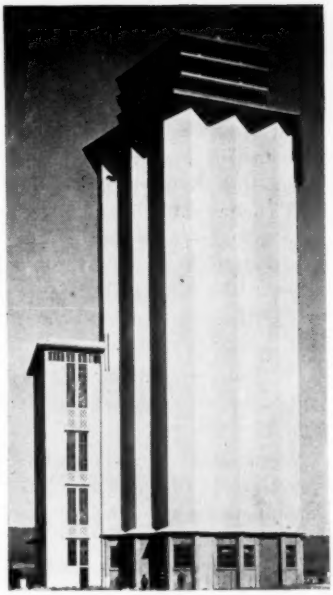


**Fig. 1—Hexagonal grain elevator in Fez, French Morocco**

*Courtesy of Establissements Fourre and Rhodes, Engineers-Contractors, Paris, Algiers, Tunis and Casablanca*

vator. This layout implies a centrifugal system of grain distribution. The nominal capacity is 6000 metric tons (about 215,000 bushels).

The engineers and contractors "never had to go back to the site," as they put it; which means that the structure gave satisfaction during its 18 years of service.

It might be of interest to note that the contract was awarded, according to a common procedure under French legislation, after "concours," meaning that the design, its technical justifications and the total cost were considered the most advantageous among several competitive bids presented by qualified firms previously selected by the authorities to enter the competition. This may be considered as a successful practical test for this type of bin design.

## Leaching of Lime from Concrete (LR 46-48)*

C. T. Wolley, Freeman, Fox and Partners, London, England, cites experience with the leaching of lime in a concrete arch dam.

I was much interested to read the two letters* in the February issue of the Journal regarding the leaching of lime from concrete and the resultant strength loss.

*See also ACI Journal, Feb. 1950, *Proc.* V. 46, p. 473.

I had some experience with this leaching action in a small concrete arch dam in North Wales built about 25 years ago. At that time the control of concrete was much less rigid than it now is. Throughout its life there has been a slight percolation of water through the concrete of this dam commencing about 10 ft below the top and becoming more pronounced towards the lowest levels, where the head is 90 ft. The percolation shows itself as a dampness on the back face of the dam, and is at such a rate that it is generally balanced by evaporation, but this process leaves a deposit of lime (and other impurities) on the surface. The deposit is irregular and unsightly and has been removed several times. The total measurable leakage, including percolation, at the base of the dam is about 2 gpm, the bulk of which comes from one small leak in a rock abutment.

The water stored behind the dam is a moorland water, free from lime, containing an extremely small percentage of peaty acid and probably some free $CO_2$.

A few years ago I became concerned at the continued removal of lime from the concrete and the accompanying effects. Arrangements were made for cores to be taken from the concrete and tested. Cores were drilled horizontally from the back face at various points and extended to within one foot of the water face of the dam. These cores were examined at an analytical and testing laboratory.

From visual inspection, the cores, which were 2¼ in. diameter, showed no apparent deterioration of the concrete. One of the samples had been cut through a construction joint without failure on the joint. All the samples appeared to be dense concrete and there was no evidence of the percolation.

The analysis showed that the proportion of cement to sand varied between 1 to 2.5 and 1 to 3.1, the richer mix being near the front face. The analysis of one of the samples was:

| | Percent |
|---|---|
| Sand | 66.44 |
| Silica | 4.47 |
| Alumina and iron oxide | 2.95 |
| Lime | 13.45 |
| Magnesia | 0.36 |
| Sulfuric anhydride | 0.34 |
| Loss on ignition | 11.99 |
| | 100.00 |

The chemists reported that as the lime had reacted with the silica, it was not possible to give exact figures for the loss of lime, but in no sample was there any "serious loss." In view of the layers of deposit cleared from the back face of the dam, this was unexpected, but reassuring.

Compression tests were made on samples sawed and ground to parallel faces with a ratio of length to diameter equal to 2. The average crushing strength on the basis of the equivalent 6 in. cube strength was 370 tons per sq ft. Taking into account the drilling, sawing and grinding processes to which these samples had been subjected, there was no serious loss of strength.

By the "low head" test apparatus available in the laboratory, no permeability could be detected in the samples.

The experience noted above is of interest only in recording that it is possible to have visible evidence of continued heavy leaching without any serious accompanying changes in lime content—or what really concerns the engineer—the strength of the concrete.

# Current Reviews*

## of Significant Contributions in Foreign and Domestic Publications

### Analysis of flexible or rigid reinforced concrete arch bridges reinforced with stiffening beam (Il caicolo dell'arco flessibile o rigido con trave irrigidente)

GIUSEPPE RAINERI, *Giornale del Genio Civile* (Rome), V. 87, No. 3, Mar. 1949, pp. 132-137

Reviewed by GENNARO MIANULLI

The author discusses the analytical analogy between a flexible and a rigid arch limited to the vertical plane containing the arch and stiffening beam.

### Influence lines for indeterminate structures (Teoria delle combinazioni lineari)

CARLO CATALANO, *Giornale del Genio Civile* (Rome), V. 87, No. 3, Mar. 1949, pp. 116-118

Reviewed by GENNARO MIANULLI

The author develops influence lines for reinforced concrete continuous bridges. Bettis theorem is used to demonstrate the theorem of line combinations and coefficient constants are worked out for the elastic deformations of beams under consideration.

### Creep, shrinkage and plastic flow in a reinforced concrete structure (Effectio statico delle distorsioni cicliche in regime viscoso)

CESARE CASTIGLIA, *Giornale del Genio Civile* (Rome), V. 87, No. 3, Mar. 1949, pp. 132-137

Reviewed by GENNARO MIANULLI

The author attempts to analyze the influence of retarding agents on the statical behavior of a young reinforced concrete structure. Formulas are developed and their effects discussed.

### Concrete manual

U. S. Bureau of Reclamation, Fifth Edition, 1949, 489 pp. For sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., and the Bureau of Reclamation, Denver Federal Center, Colo. $1.75

Keeping pace with recent developments in concrete technology, the fifth edition of this widely-known manual has new material on prospecting for pozzolanic materials, the manufacture and acceptance of precast concrete pipe, sawdust concrete, prepacked concrete and vacuum-processed concrete. The chapters on concrete mixes, concrete manufacture and handling, placing, finishing and curing have been revised or expanded. An added chapter on repair and maintenance describes new methods in this field.

While primarily intended for the use of Bureau engineers, the scope of this manual makes it useful to anyone interested in concrete work.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 9, May 1950, *Proceedings*, V. 46. Address 18263 W. McNichols Rd., Detroit 19, Mich. Copies of articles or books reviewed are **not** available through ACI. In most cases they can be obtained direct from the original publishers. Address, when available, will be furnished by ACI on request.

### Determination of the ultimate moment of prestressed concrete beams (Die Ermittlung des Bruchmoments von Spannbetontragern)

E. Mörsch, *Die Bautechnik* (Berlin), V. 26, No. 4, p. 98 — Reviewed by Rudolph Fischl

This article gives a graphical solution for the determination of the ultimate moment in prestressed reinforced concrete beams. The stress-strain diagram has to be known from tests for both materials up to the point of failure. Both cases, failure of the compression zone and failure of the prestressed reinforcement, are discussed and illustrated.

### French-English technical glossary of engineering equipment terms (Lexique technique—materiel de travaux publics)

Institut Technique du Batiment et des Travaux Public, Paris, 184 pp. 700 F

Translation of technical engineering terms often is difficult using an ordinary bilingual dictionary. As an aid to those ordering plant and spare parts or operating, maintaining and repairing equipment described in catalogs in the opposite language this French-English glossary was prepared. Detailed conversion tables facilitate conversion from English to metric units or vice versa.

### Theory and practice of construction (La teoria e pratica delle costruzioni)

G. B. Ormea, Milano, 1945, V. 1—Statica e resistenza del costruzioni civili, industriale e rurali
Mario Corsetti, Paravia Torino, 1945, V. 2—Civili, stradali, idrauliche etc. — Reviewed by Gennaro Mianulli

These textbooks treat in very simple language the elements of statics, resistance of materials and construction methods for highway, railroad, hydraulic and rural structures. These texts are in accordance with the curriculum of technical high schools in Italy.

### Methods of graphing several variables

J. D. McIntosh, *Magazine of Concrete Research* (London), No. 3, Dec. 1949, pp. 145-148 — Author's Summary

A description is given of methods of plotting experimental values of more than two variables in such a way that relationships between each pair of variables can be established taking all experimental values into account; these methods are illustrated by examples taken from concrete research. Their use permits a considerable reduction in the amount of experimental work involved in examining the effects of several variables on a particular property.

### Concrete as a protective barrier for gamma rays from cobalt-60

Robert J. Kennedy, Harold O. Wyckoff and William A. Snyder, *Journal of Research*, V. 44, Feb. 1950 — Reviewed by C. C. Fishburn

Data based on the experimental investigation of broad- and narrow-beam attenuation of $Co^{60}$ gamma rays are presented. In protective barriers the geometry necessarily requires the use of broad-beam attenuation curves to determine the required barrier thickness. This paper presents attenuation curves for lead, concrete, and steel necessary for determining the required protection for any particular cobalt source in storage or transportation. Also included is a table for shielding requirements for $Co^{60}$ sources of 10 millicuries to 2 curies.

### An index of nomograms

Compiled and edited by Douglas P. Adams, John Wiley & Sons, New York. $4.00

This book gives the location and content of 1700 alignment diagrams published extensively in current technical journals. Emphasizing the use of nomograms in

practical engineering problems requiring the repeated use of the same equation, the index covers a wide range of fields. Among these are: chemistry and chemical engineering; mathematics; physics; electricity, electronics and radio; hydraulics and power; aeronautics; waterworks and sewage; illumination; heating, piping and ventilating; oil and gas; building and surveying; mining; machine tools and design; metals, textiles; medicine; food; and transportation.

### Roof trusses in precast reinforced concrete (Stahlbetondacher in Fertigbauweise mit Fachwerkbindern)

K. Koznietzki, *Die Bautechnik* (Berlin), V. 26, No. 4, p. 99 Reviewed by Rudolph Fischl

Three precast roof constructions, recently erected by Beton and Monierbau A.-G. Berlin, are described.

The T-shaped precast chord members were lifted in place at the site, temporarily supported, and the joints filled with concrete.

The span of the two-panel trusses varies from 36 to 44 ft with distances between the bents from 5 ft 7 in. to 13 ft 2 in. on center. Concrete of quality B 225 and plain reinforcing steel (Beton-stahl I) were used. The quantities of material for one truss including purlins per sq ft of area varied from 0.115 to 0.213 cu ft of concrete and 2.25 to 3.18 lb of reinforcing steel.

### Engineer's dictionary (Spanish-English and English-Spanish)

Lewis A. Robb, John Wiley and Sons, Inc., New York, 2nd Edition, 664 pp. $12.50

To the engineer who wishes to translate engineering terms or phrases from English to Spanish, or vice versa, this book is an invaluable aid to finding the correct shading of meaning of word combinations common to engineering but ordinarily not found in the usual dictionary. Except where necessary to avoid confusion definitions of words are not given, attention being concentrated on authoritative technical equivalents. Civil engineering terminology has been brought up to date and new material on electrical and mechanical engineering, sparsely represented in the first edition, has been added. Such terms as air-entraining, bank-run gravel, batch meter, bleeding test, coarse aggregate, lean concrete, prestressed, random cracking and water gain with their Spanish equivalents are but a very few of those related to concrete to be found in this dictionary.

### Long duration impulsive loading of simple beams

R. J. Hansen, *Boston Society of Civil Engineers Journal*, V. 35, No. 3, 1948, pp. 272-85 Building Research Abstracts Apr. 1949

In order to design reinforced concrete structures capable of resisting impulsive loading of short duration, the allowable stresses for concrete and steel have been determined experimentally. For short duration pulses these allowable stresses are about ten times those allowable for static stresses. In this paper consideration is given to impulsive loading of long duration and the permissible design stresses therefor. A description is given of the design and construction of the impact testing machine used, the instrumentation procedures adopted in connection with its operation, and some very tentative results so far obtained for beams (53 x 4 x 5¼-in. deep) subjected to just cracking loads. Data suggest that the concrete and steel will stand stresses considerably in excess of the respective yield and failure stresses. The concrete in the various beams tested sustained stresses from 1.1 to 2.5 times the ultimate strength of the concrete, while the steel sustained stresses ranging from 2.5 to 3.5 times the static yield stresses.

### The relation between strength and free water content of aerated concretes

G. E. Bessey and S. Dilnot, *Magazine of Concrete Research* (London), No. 3, pp. 119-122

Authors' Summary

The effect of progressive small increments of moisture upon the strength of completely dry aerated concretes has been examined. The decrease in strength due to the first 2 to 3 percent of moisture has been found to be considerable, and to constitute a high proportion of the total drop in strength from the dry to the completely wet condition. When expressed as a percentage of the dry strength, the drop in strength of aerated concrete from the completely dry to the completely soaked condition does not differ markedly from that of ordinary dense concrete.

Since it is possible that the strength of dense concrete is similarly affected by small contents of free moisture, it seems that comparison of dry strength test results can have little value without a precise definition of the degree of dryness.

### Slab bridges (Plattenbrucken)

R. Ohlig, *Die Bautechnik* (Berlin), V. 26, No. 4, pp. 118-121

Reviewed by Rudolph Fischl

With the allowable stresses for concrete of quality B 300, continuous slab bridges are economical for spans up to 82 ft and even more.

For slab bridges with small spans the simplified design method given in the German specifications may be used. For larger spans, however, it is desirable to make a careful analysis covering the influence of unsymmetrical and concentrated loads. By means of the partial differential equation of the fourth degree for the deflection of slabs the moments parallel and perpendicular to the main span and the torsional moment can be computed. Influence lines for moment values in different sections are illustrated, and the effect of dynamic forces on the vibration of the bridge and construction viewpoints are discussed. A section through a slab bridge with three spans approximately 78 ft long is shown.

### Analysis of thin arch bridges reinforced by stiffening beams (Sul calcolo statico dei ponti adarco sottile con trave irrigidente secondo la teoria del II ordine)

Pedro B. J. Gravina, *Giornale del Genio Civile* (Rome), V. 87, No. 5, May 1949, pp. 207-216

Reviewed by Gennaro Mianulli

This paper deals with the analysis of thin arches reinforced by stiffening beams and wide spans. The influence of the deformation on the structural system is taken into consideration. This analysis is analogous to that used for suspended bridges. There is a difference only in the direction where the effect of the deformation takes place.

Trigonometric series are employed to investigate the displacement and also to obtain the expressions for the moments in the arch and in the stiffening beam. Then the equation for the evaluation of the additional thrust in the arch is deduced. An example illustrates the application of the theory and an easy method is given to determine the value of the additional thrust eliminating trial and error procedure which is usual for structures of this type.

### Moments in two-way concrete floor slabs

C. P. Siess and N. M. Newmark, *University of Illinois, Engineering Experiment Station, Bulletin* 385, Feb. 1950, 124 pp. 60 cents

Authors' Summary

The development of an approximate moment-distribution procedure for the analysis of uniformly loaded plates continuous in two directions over rigid beams is described in detail. The procedure is then used to study the effects of several important variables on the moments in two-way slabs.

The moment-distribution procedure and the results of the analyses were included in the paper "Rational Analysis and Design of Two-Way Concrete Slabs," by the same

authors, in the ACI JOURNAL, December, 1948. This bulletin furnishes the background and detailed development of the procedure, and presents the results of the analyses in full.

Exact solutions for moments in rectangular plates with various edge conditions are presented in three appendixes.

### The vibration of concrete slabs

R. H. H. KIRKHAM, *Magazine of Concrete Research* (London), No. 3, Dec. 1949, pp. 141-143

An experimental vibrating machine used to study the variables in the compaction of lean dry concrete in concrete pavement slabs consists of a hollow screed containing the mechanism for producing vibration and shod with a removable shoe. It is hung by anti-vibration supports from an overbridge carried on rails outside the forms. Vertical, horizontal, inclined or rotary vibration may be produced.

Under test the machine had a controlled frequency range of 500 to 7000 vibrations per minute, a controlled amplitude range of 0.004 in. to 0.075 in. and a forward speed of 1 to 16 fpm. For preliminary tests the machine is being used to compact a concrete slab 15 ft long, 4 ft wide and 18 in. thick after passage of the machine. It is hoped to determine optimum vibration conditions which would lead to the design of cheaper, more rugged and more efficient surface vibrators.

### Design of reinforced concrete cylindrical shells (Beraekning av cylindriska skal med haensyn till den armerade betongens egenskaper)

G. DE KAZINCZY, *Betong* (Stockholm), V. 34, No. 4, 1949, pp. 239-261 AUTHOR'S SUMMARY

Cylindrical shells of sufficient length $l$ in relation to their width $b$ (*i.e.*, $l \geq 2b$) may be considered as beams. Their load-bearing capacity can be determined with greater accuracy if the assumption of an elastic and homogeneous material is replaced by the assumption of a cracked section as usual in the design of ordinary reinforced concrete structures. The calculation of longitudinal and shear stresses and of bending moments at right angles to the generating lines by a method of finite differences is explained in detail and demonstrated by an example. The difference between the results obtained by the two methods (*i.e.*, elastic and homogeneous material on the one hand and cracked section on the other) is shown. Special cases such as single shells with a symmetrical loading, external bays of a series of continuous shells and certain special cases of loading on round silos will be dealt with in a later publication.

### Mechanism of the effect of air-retaining substances on portland cement mortars and concretes

YU. M. BUTT and T. M. BERKOVICH, *Journal of Applied Chemistry* (U.S.S.R.) V. 22, No. 7, 1949, pp. 653-60, *Doklady Akad. Nauk S.S.S.R.* V. 60, No. 9, 1948, pp. 1551-54 CERAMIC ABSTRACTS Feb. 1950

The addition of rosin and abietic soaps retarded the sedimentation of cement suspensions; the peptization effect produced by abietic soap was greater than that produced by rosin soap. Both soaps raised the plasticity of cement paste; the addition of 0.03 percent soap by weight of the cement increased the plasticity 35 to 45 percent. The soaps had no adverse effect on the strength during the first 7 days, but after 28 days 2 percent of either soap reduced the strength 20 to 30 percent; the strength was affected by abietic soap to a lesser degree than by rosin soap. The addition of the soaps increased the true porosity of the hardened cements, the increase being caused by a rise in the number of closed pores. The various effects of these soaps are explained by the mechanism of the absorption processes; they follow equations similar to that of the isotherm of absorption.

### End anchorage and bond stress in prestressed concrete

G. Marshall, *Magazine of Concrete Research* (London), No. 3, Dec. 1949, pp. 123-27

Author's Summary

This paper deals with the results of tests at Leeds University on prestressed concrete columns to find the transmission lengths required to ensure complete bond between the wires and the concrete.

It is seen that the initial transmission length for 0.2 in. diameter wires stressed to 70 tons per sq in. is about 125 diameters (25 in.) and of 0.08 in. diameter wires stressed to 100 tons per sq in. about 60 diameter (5 in.).

These values apply only if the concrete is of good quality. It is seen that bad placing can cause these transmission lengths to be greatly increased. A transmission length of approximately 400 diameters was measured in badly placed concrete.

It is recommended that some form of mechanical locking device be used when structures of short length are to be prestressed by 0.2 in. diameter wires.

The use of 0.08-in. wires eliminates any uncertainty with regard to the end anchorage.

### Wood-wool concrete

S. Geleff, *Indian Concrete Journal*, V. 22, No. 12, 1948, pp. 271-4

Building Research Abstracts Apr. 1949

The manufacture, properties and uses of wood-wool concrete building slabs are described. The slabs, about 1½ to 4 in. thick, are made by either the wet or the dry process, the chemically treated coniferous-wood fibres being mixed with either magnesite, plaster, or portland cement, and pressed (screw or hydraulic) in molds to the required thickness. Resistance to moisture, frost, rot, and fungi is very satisfactory, binding strength increases with increasing specific weight, while the fire resistance and the virtual noninflammability of the slabs are favorable to their use in lightweight buildings. The coefficient of heat transmission is 0.57. The slabs are used mainly for heat insulation on the inner side of brick or concrete walls and in roof structures, especially in hot climates or where the temperature changes over a range of 80 F, *e.g.*, Denmark. They are also used as permanent form work for reinforced concrete floors and roofs, concrete walls, and also as double- or single-thickness partition walls. The sound-insulating value of the slabs is reflected in their use as subfloors for floated concrete floors, and as acoustic linings and have been so used for house construction in Sweden, Denmark, and Germany.

### The stress distribution near the line of contact between triangular gravity dam body and foundation

Tojiro Ishihara and Yoshiji Niwa, *Journal of the Civil Engineering Society of Japan* (Tokyo), Special Issue Collection of Treatises 1947, 1948

Reviewed by S. Takahashi

Previous analytical elastic studies of the stress in the interior of a dam chiefly were based on a semi-infinite triangular section. They, consequently, were unable to evaluate the stress distribution near the line of contact between the dam body and foundation.

In this paper the stress distribution near the line of contact has been obtained by modifying the stress in the interior of a semi-infinite triangular dam by a revised stress function and correlating the displacement between the dam and foundation along the line of contact with various constants in the revised function. The result was that with the stress distribution along a horizontal line of contact, as the water rose to the top of the dam, the water pressure and the weight of the dam itself gave no rectilinear change but there was a large effect due to the elastic nature of the dam body and foundation.

This emphasizes the importance of taking a rational point of view with a high dam by ignoring the effect on the foundation and stressing the importance of the usual method of computing elasticity of semi-infinite triangular sections.

### The economic design of rectangular reinforced-concrete sections

T. P. O'Sullivan, *Institution of Civil Engineers Journal*, V. 32, No. 6, 1948, pp. 175-205

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Factors arising in the design of rectangular reinforced concrete sections to resist stresses due to compressive or tensile loads applied normal to the section and at a point along the line of its major or minor axis are investigated. The optimum arrangement of reinforcement in such sections is developed from the economic standpoint. It is concluded that symmetrical reinforcement may prove very extravagant in rectangular sections since, with a compression load, the steel required may reach three times the minimum, while with a tensile load it may reach double the minimum. An approximate economic solution for such sections was developed by Dumontier in 1928. The author has developed exact equations representing the special economic solution whereby, if a compression load is on the edge of the section, there is a saving of 20 percent in reinforcement over the usual economic design involving maximum permissible stresses in both steel and concrete. These equations are claimed entirely to eliminate trial and error methods in design when determining the reinforcement required to stabilize a given section. Use of high-tensile reinforcement with low-stress and high-stress concrete has no advantage in cases of bending and compression, simple bending or bending and tension, involving high eccentricity ratios, except where light reinforcement only is required for use with high-stress concrete.

### The strength of concrete and its measurement (La resistance du beton et sa mesure)

R. L'Hermite, *Annales de L'Institut Technique du Batiment et des Travaux Publics*, No. 114, Jan. 1950, *Beton, Beton Arme*, No. 12, 19 pp.

This paper is both a sequel and a reply to discussion of a lecture by the author (R. L'Hermite, "La Resistance du Beton et sa Mesure (The Strength of Concrete and its measurement), *Annales de L'Institut Technique du Batiment et des Travaux Publics*), on the latest research work at the Laboratories du Batiment et des Travaux Publics on the manufacture and control of concrete mix and likely future developments. The following points were dealt with: the possiblilty of considerably improving the normal tests of the mechanical strength of concrete, whether on the site or in the laboratory; concrete mixing and the control of homogeneity, a nondestructive method of measuring the mechanical properties of concrete and reinforced concrete, etc.

The replies given cover the quality of cement, the effect of different mixes on strength, the manufacture of test specimens, whether the concrete of the specimen truly represents the finished structure, the effect of storage conditions on the strength obtained, the effect of the size of the specimen, crushing tests, the shape of the test pieces and discussion of the heterogeneity of concrete and its effect on strength.

In conclusion, the author outlines the program of work which remains to be done on modification of acceptance standards of concrete, improving the composition of concrete, and standardization of manufacture, storage and crushing tests of test pieces.

### Measurement of particle size and specific surface of cements (Mesure de la granulometrie et de la surface specifique des ciments)

Jacques Brocard, *Annales de L'Institut Technique du Batiment et des Travaux Publics*, No. 113, Jan. 1950, *Liant Hydrauliques*, No. 6, 27 pp. Author's Summary

Measurement of particle size by counting by microscope is a lengthy and difficult process and not a practical proposition; it does, however, serve as a standard for other

types of apparatus. Sifting gives results which are correct for particles up to about 90$\mu$. The most important practical results are obtained by sedimentation; and the best conditions for carrying out experiments can be found by a critical study of the phenomenon.

A number of types of apparatus are criticized, and the Andreasen pipette is judged to give the most accurate results. The Wagner turbidimeter gives quicker results but it is less accurate than the pipette. Flourometry is useful for obtaining quickly the shape of the grading curve and is proving a valuable method of control in industry.

Where particles are assimilated into spheres, sedimentation curves enable one to calculate the specific surface of cement by allowing for an average diameter for each fraction. Measurement is quickly made with a turbidimeter.

Methods which consist in passing a fluid through the compressed powder are even more rapid and for this reason useful in controlling crushers. The principle seems good for checking artificial portland cements. On the other hand, less accuracy is reached in the control of cements of various origins.

Adsorption methods take into account all the roughnesses of the particles and produce accurate results approximating closely the real specific surfaces but the processes are delicate and lengthy.

### Plate and rib floorings (Sul calcolo delle piastre e nervature)

CARLO TOLOTTI and GIUSEPPE GRIOLI, *Giornale del Genio Civile* (Rome), V. 87, No. 6, June 1949, pp. 275-308 Reviewed by GENNARO MIANULLI

This paper treats the grid system of reinforced concrete construction, consisting of square or rectangular slabs supported by marginal beams and reinforced in two directions. Marginal beams placed diagonally also are given attention.

The theoretical analysis for this type of floor system is not simple. Various attempts have been made to solve the problem by approximate methods. The formulas in use are not consistent with results obtained by tests. An exact theoretical analysis is very complicated and impractical.

This investigation was sponsored by the National Institute of Applied Mathematics (Istituto Nazionale per Applicazioni del Calcolo) and the two authors as members of its research staff attempted to arrive at a rational and practical method of solution.

The assumptions are (1) the load distribution from slab to its reinforcing ribs is questioned, (2) a study is made of the statical behavior of the entire slab-rib system by separation of the slab and rib and establishing for each an equation of elastic equilibrium, (3) this separation of the two members is allowed only if the actions exerted by the slab along the edges of the ribs and vice-versa are taken into consideration, (4) although these actions are not evident (under their influence each rib remains in equilibrium) they are translated into convenient differential expressions at the unknown elastic displacements of the rib, (5) the study of the elastic equilibrium equations for the slab's unknown displacements are expressed by integrals operating on the forces which act on the slab, with forces developed in the rib included and (6) a system of integro-differential equations are established to compute the unknown displacements of the rib, by combining the elastic displacements of the slab and rib according to the condition of contact between the two members.

The analysis was made first for a slab simply supported on the ribs. With some modification the same equations developed for the first case were applied to slab and rib casts monolithically and acting as a T-beam.

The authors thought was to express, merely, the tangential actions between slab and rib by means of the deflections of the rib only. The ordinary theory of flexure for beams was used and a convenient width of slab acting integrally with the rib was considered.